THE ENGINEERING RECORD

PUBLISHED WEEKLY BY THE

McGraw Publishing Company

James H. McGraw, Pres.
J. M. Wakeman, 1st Vice-Pres.
A. E. Clifford, 2d Vice-Pres.
C. E. Whittlesey, Sec. and Treas.

John M. Goodell, Editor.

MAIN OFFICES:
239 WEST 39TH STREET, NEW YORK.
BRANCH OFFICES:
CHICAGO: Old Colony Building.
PHILADELPHIA: Real Estate Trust Building.
CLEVELAND: Schofield Building.
LONDON OFFICE: Hastings House, Norfolk St., Strand.

SUBSCRIPTION RATES.

United States, Cuba and Mexico............$3.00 a year
Canada ....$4.50 a year
Foreign subscription ....$6.00 a year
25 shillings. 25 marks. 31 francs.
Foreign subscriptions may be sent to our London office.

NOTICE TO ADVERTISERS.

Changes in advertisements intended for a particular issue should reach the office of THE ENGINEERING RECORD on Friday of the previous week. New advertisements can be received up to noon of Wednesday of the week of issue.

NOTICE TO SUBSCRIBERS.

REMITTANCES.—Remittances should be made by check, New York draft, or money order, in favor of THE ENGINEERING RECORD.

CHANGE OF ADDRESS.—The old address should be given as well as the new, and notice should be received a week in advance of the desired change.

BACK COPIES.—No copies of issues prior to January, 1907, are kept on sale, except in bound volumes.

DATE ON WRAPPER indicates the month at the end of which subscription expires. The sending of remittances for renewal prior to that date will be much appreciated by the publishers.

CLUB RATE.—In acknowledgment of assistance rendered by subscribers in the securing of new orders, a club rate of $2.50 per annum is quoted. This rate applies to subscriptions in clubs of two or more, at least half of the names in the club to be those of new subscribers.

Entered at the New York Post-Office as Second Class mail matter.

Copyright, 1908, by the McGraw Publishing Co.

The circulation of THE ENGINEERING RECORD for 1907 was 763,000 copies, an average of 14,673 copies per week.

Of this issue 14,000 copies are printed.

LIST OF LEADING ARTICLES

Tests of Columns in Connection with the Quebec Bridge Failure.................................... 513
An Analysis of Road Conditions.................. 513
The Protection of Lake Michigan from Contamination. 514
The New Four-Track Entrance of the Erie Railroad Into Jersey City (Illustrated).................. 516
Cost of Sand and Gravel for Gunnison Tunnel Lining 519
The New Steel Rail Specifications of the Pennsylvania System (Illustrated).......................... 519
A 25,000-Horse-Power Reversing Engine for a Blooming Mill (Illustrated)......................... 521
A Continuous Unloader............................ 521
Subsurface Investigations on the Catskill Aqueduct, Board of Water Supply (Illustrated)............ 522
The Canal Street Tunnel Sewer, New York City, (Illustrated) 525
A Light, Long Steel Derrick Boom (Illustrated).... 527
Water Softening at Oberlin, Ohio................. 528
Appendices 15 and 16. Report of Royal Commission on Quebec Bridge (Illustrated).................. 529
A High Pressure Water Works..................... 539
A Deep-Water Dredge Bucket (Illustrated)......... 539
Book Notes....................................... 540
The National Forests in the White and Southern Appalachian Mountains........................... 540
Chrome Pigments for Paints....................... 540

Tests of Columns in Connection with the Quebec Bridge Failure.

The printing by The Engineering Record of the complete report of the Royal Commission appointed to investigate the causes of the failure of the Quebec Bridge affords an unusual opportunity to discover the action of great steel compression members under loading, both in the structure itself and under test in the large testing machine at the works of the Phoenix Bridge Co. This is particularly true of the testing to failure of the model of the lower anchor-arm chord section in which it has now been established the initial failure of the structure took place. The detailed account of this model of the actual chord section will be found on another page of this issue. Although this column was a model only of the actual chord panel of the bridge, accurately constructed on a linear scale of one-third, its area of normal section was 86.5 sq. in., or one-ninth the area of section of the full-size member, it was far larger than any metal column ever before placed under test and the results obtained in the investigation are correspondingly valuable.

The full report gives a clear account of every feature developed by the increasing loads. Fortunately the plan of conducting the test involved the observing and measuring of any deflection of the column which might take place, or any other evidence of approaching collapse of the member. It was known in advance that the comparatively light latticing would not afford sufficient resistance to develop the full carrying capacity of the four webs of the column constituting its entire effective section. While it was anticipated by the members of the commission that there might be possibly some initial buckling of the web plates, the real object of the test was the determination of the spacing capacity of the angle lattice bars and their connections. This was the more important as the ratio of length to radius of gyration is at the lower limit of application of ordinary column formulas, this ratio being 25 in one direction and 42 in the other. With such a short column it could scarcely be expected that any material amount of bending would take place prior to failure.

As a matter of fact, the report of the test shows that the column carried its load up to an actual stress of about 22,110 lb. per square inch and then instantly failed by the shearing of a large number of the rivets securing the lattice angles to the flanges of the webs. The extreme suddenness of the failure with no sensible distortion of any of the details of the column is interesting and significant. The commission appears to believe that there may have been some shearing distortion of the lattice rivets not discoverable by the insufficient method of measuring center line distances along the lattice bars between prick punch marks on the heads of rivets, but that seems inadmissible from the fact that there could have been no material shearing distortion of the rivets without corresponding displacement or deflection of the web or webs, and no such deflection was discovered. As stated in the report, the entire column failure took place under elastic deformations of the main elements of the column.

As there was no buckling or other material distortion of the webs, it is obvious that the results of the test as that of a column were not very fruitful as a whole, but as bearing upon the purpose for which the test was made, the results are of the highest possible value. When due allowance is made for the friction developed in the cylinder of the hydraulic testing machine and for the enhanced physical qualities of the small lattice rivets, the ultimate carrying capacity per square inch of the model column was found to be practically the same as that of the lower chord member of the anchor arm in which the failure of the bridge began. In other words, the test has shown conclusively why the bridge failed and in what respect the design was so defective as to produce that failure. The plan of testing the model chord and the method of carrying out the investigation so as to produce results indicating clearly the source of weakness of the member and so analyzing the data thus secured as to reach a conclusive determination of the question at issue, constitute what may be considered the most finished and satisfactory structural investigation by large-scale testing ever accomplished.

The short column consisting of but two webs, also tested at Phoenixville, was intended to exhibit the carrying capacity of the column webs. While the results are interesting they are unsatisfactory in many ways and scarcely conclusive in any respect. The column was so short that there was room for only three panels of latticing between the end battens; in fact, if it had been a little shorter the test might almost have been made with no latticing at all. This exceedingly short column failed in detail, as was to be expected, by the bending of the webs between the panel points, and not, strictly speaking, by the buckling of the web plates. The latter expression used in the report was either an oversight or meant to indicate the bending of the web, as stated above. The riveting at the end of the lattice bars was made double that in the model column and no failure of the latticing took place, although the ultimate load was about 37,000 lb. per square inch. The extreme shortness of this column prevents any useful conclusion being drawn from the results of the test in regard to the latticing. Again the fact that two webs only were used, instead of the four in the full-size chord, would prevent a proper comparison being made between the two tests.

Now that the circumstances of the failure of the structure have been established, it is of the utmost importance that full-size column tests shall be made similar to that of the model, with increasing size of lattice bars and capacity of riveting, so that safe rules of design for steel columns may be established for built compression members spaced either with latticing or cover plates. It would not be surprising to find that the elastic limit of the material coincides nearly with the ultimate carrying capacity of columns.

An Analysis of Road Conditions.

At a recent meeting of the Royal Society of Arts, a paper on the problem of road construction with a view to modern and future requirements was read by Dr. H. F. Hele-Shaw and Mr. Douglas Mackenzie that deserves notice from American engineers. Dr. Hele-Shaw is probably best known on this side of the Atlantic as the inventor of a multiple disc clutch for automobiles which is rapidly coming into favor. Lest readers who are acquainted with him only through this invention should be misled into the belief that he is an automobile enthusiast who looks upon roads solely as a means of taking drives for pleasure, it should be stated that he is one of the leading engineering authorities of England, where his research work in many lines has been noteworthy. His paper is interesting inasmuch as it takes up the road problem from a new point of view, or at least from a point of view that is rarely considered. In most discussions of road work, it is customary to begin with drainage details and the foundations on which the roadway itself is constructed. In the paper this procedure is reversed, and the author approaches the subject from the point of view of a mechanical engineer whose object is to determine the best machine for a given purpose, the machine in this case consisting of the wheel and road. The old methods of construction produced roads which were satisfactory for slow-moving vehicles, but they are not satisfactory for motor cars, and it is probable that when motor wagons also become common the troubles incident to keeping the roads in condition will be multiplied. The expense of road construction and maintenance in Great Britain has increased so rapidly in the last ten years that the engineers and authorities responsible for the work have become alarmed, and rightly so, for such expenses are mounting up at a rate which shows that it is absolutely necessary to take some steps to prevent their further increase.

If the road and the wheel are regarded as parts of a machine, it is self-evident that their ideal combination exists when a perfectly hard and circular wheel runs on a perfectly hard and level road. The steel wheel and rails of a railroad have been developed on this principle, but it is impracticable, of course, on any highway. No road surface is truly level, and to mitigate the shocks due to its unevenness rubber tires have been found more useful than anything else. By the adoption of the rubber tires, however, the character of the machine changes. As Dr. Hele-Shaw points out, we cannot employ this soft material without paying the penalty, not merely of wearing the wheels, but of wearing the road itself. Just so far as the contact between the wheel and the road departs from a line, by so much the wear between the surfaces in contact is increased. If the road is not absolutely hard a certain amount of deformation takes place and the injury due to this deformation will depend upon the depth to which it extends and the extent of the permanent disintegration of the internal substance of the road. Inasmuch as rubber tires have come to stay and they are the new feature of road traffic which seems to be causing most trouble, it seems logical that the road of the future should resemble the rubber tire in being of a tough, elastic material, or since that is probably impracticable, the material of which the road is mainly composed will be bound together by some fairly elastic and tough substance. This construction will be waterproof, free from mud and free from dust. The object should be to produce a road thoroughly bound together and so hard that the surface of contact between it and the wheel passing over it is reduced to a minimum. This last condition is desirable because observation and experiment have both shown that wear increases in proportion to the amount of surface which is in contact.

It is hardly necessary to follow the paper under consideration through the résumé it contains of the experiments with dust preventives. The value of these substances sprayed over the surface of the road is a matter of debate in this country, as in other countries, and while they unquestionably generally produce some temporary effect it is still uncertain how long their application remains of use. Dr. Hele-Shaw is of the opinion their cost is so great as to render them unserviceable for most places. His investigations of tar, macadam and similar methods of construction have convinced him that such materials are too expensive for use in most English districts, where the road taxes are already as heavy as the people can carry. Such systems of construction, he states, are estimated to cost, as a rule, at least twice as much as roads constructed of the same thickness in the ordinary way and although their first cost is justified in a measure by the increased life of the road, if a district is financially unable to raise the money for such high first costs, the system of construction is out of the question.

The situation is not a serious one, however, in his opinion, on account of a method of construction devised by Mr. A. Gladwell. This he regards as the solution of the road problem of the future, although it may be necessary to modify it in details. Mr. Gladwell uses a tar preparation, several of which are on the market in Great Britain, to make a matrix of fine granite chips mixed in this tar preparation. This matrix he lays on the old surface of the worn road to a depth of ½ to ¾ in. He then spreads the broken road stone on the top of this to a thickness not exceeding 4 in., and rolls it immediately with a steam roller. The latter forces the stone down into the matrix, which is squeezed up into the interstices. If the materials have been laid in the right proportions, a little rolling will bring the matrix almost up to the surface. A small quantity of the matrix is then laid over the surface and brushed in, and a final rolling is given it in order to produce a thoroughly waterproof surface. The cost of this construction is from 4 to 6 cents per square yard greater than the cost of a road surfaced with the same amount of stone, but built with the aid of a water sprinkler in the usual way. Whether this system of construction is as great an improvement as Dr. Hele-Shaw's enthusiastic comments on it would indicate can only be determined by the test of time. It is attracting considerable attention in Great Britain, and the belief seems to be fairly prevalent that the roads built in this way will last several times as long as an ordinary road under motor-car traffic. The surface is waterproof and tough, thus promising freedom from mud and dust and satisfying as well as any road can the conditions of the theoretical machine comprising road and wheel. The system is not protected in any way by patents and, it should be added, has received the support of the Roads Improvement Association of England. One feature which experience has shown to be necessary is the thorough drying of the materials. A road of this sort built by W. H. Leete, at Bedford, which was cut open in order to determine exactly its condition, was found to have its middle third free from any of the tar. This condition was due to the fact that the stone was not dry and the work was done in the autumn, when the tar cooled off quickly before its complete penetration into the interstices had taken place. Mr. Leete believes that it is possible to build a road of this sort for 36 cents per square yard which will be far better than roads constructed with water sprinkling at a cost of 30 cents per square yard.

The Protection of Lake Michigan from Contamination.

A movement to suppress all sources of sewage and other contamination in the waters of Lake Michigan was inaugurated on April 11, in Chicago, at a conference attended by nearly one hundred representatives of more than forty municipalities situated on its shores. At this conference, called by invitation from the Chicago Association of Commerce and Mayor Busse of that city, a permanent organization was formed with Hon. Sherburn M. Becker, mayor of Milwaukee, as chairman; Dr. W. A. Evans, commissioner of health, of Chicago, vice-chairman, and Mr. W. H. Manss, of the Chicago Association of Commerce, secretary. Arrangements were also made to form a permanent committee, consisting of officials from the more than a hundred towns and cities which derive their water supplies from the lake and embrace a population of nearly 4,000,000 people. Provision was also made for an executive committee to confer with the permanent officers and to consist of two citizens from each of the four states bordering on the lake. The organization adjourned, subject to a call of the chairman for a second meeting to perfect the organization and to formulate plans for the work contemplated.

The attendance at the conference was representative of the interests concerned, as mayors and health officers of various cities around the lake, members of the boards of health of the four states bordering on the lake, and other equally prominent public officials were present, while many of the municipalities not represented sent assurance of support. The meeting was evidence of the strong bond of interest among the communities affected by the pollution of the lake, due to the geographic situation of these communities. That this interest must be a common one is vitally essential for joint action, because it is almost useless to attempt effectively to protect from contamination the different municipal supplies derived from the lake without a united effort of the population of the entire watershed tributary to the latter.

Although the meeting was called by Chicago officials, the preservation of the purity of the waters of the lake is by no means of as great moment to that city as it is to other communities dependent on the lake for a supply. As a matter of fact, the intakes from which the water supply of Chicago is derived are at present almost entirely safe from sewage pollution, and they will be even more fully protected by improvements at present definitely contemplated. On the other hand, the supplies for many other communities are seriously threatened by sewage pollution, which can only be relieved by the execution of a general plan along the lines proposed. The folly of a large city pouring sewage into the lake and then attempting to draw uncontaminated water through intakes situated at any feasible distance from the adjacent shore has been conclusively demonstrated. Even close to smaller communities it is fairly certain that water cannot safely be derived through intakes within a half-mile from the shore. Although investigations made as long ago as 1888 indicated no presence of sewage further than six miles from the shore at any time, the study of the waters of Lake Michigan conducted more recently by the late Arthur Palmer, of the Illinois State Water Survey, in conjunction with Dr. Adolp Gehrman, determined sewage bacteria at a depth of 10 ft. from the surface as far as 10 miles from shore. It is also reasonable to believe that traces of pollution may already occasionally be found at greater depths and even more distant from the shore, due to the temporary local currents caused by the extreme barometric changes to which the lake region is subject in the summer.

The practical limitations of the distance from the shore at which water intakes may be located render the removal of the sources of pollution the only remedy certain to preserve the purity of the lake. Fortunately, the newly formed orgaization has not been, and will doubtless not be, committed to any one method of avoiding the discharge of raw sewage into the lake. This lack of committal is evidenced from a resolution adopted at the meeting instructing the executive committee "to investigate the whole subject now under consideration, and to report such plan, or plans, as seem to it feasible for the disposition of sewage and offal in some way that will preserve the purity of the waters of Lake Michigan and protect them from such pollution and contamination as render them unfit for domestic purposes." The broad, comprehensive policy thus outlined, if followed strictly, will certainly enable the organization to obtain the assistance and support of all the interests involved.

While the organization is thus pledged to the action indicated by this resolution its chief activities will doubtless be directed to the support of a commission already appointed to investigate thoroughly every phase of the conditions existing in the lake, with reference to its waters as a source of domestic supply. This commission will have thirteen members, consisting as follows: One from the Corps of Engineers, U. S. A., one from the United States Public Health and Marine Hospital Service, three from the City of Chicago, one from the State of Illinois outside of Chicago, two from the city of Milwaukee, one from Wisconsin outside of Milwaukee, one from Grand Rapids, Mich., one from Michigan outside of Grand Rapids, one representing the cities of Hammond, Whiting and East Chicago in Indiana, and one from that State at large. These various members of the committee have been appointed by the officials of the different branches of government represented, the suggestion for the appointment of the commission having originated with Mayor Busse, of Chicago. The commission is to determine by original investigations the existing degree of pollution of the lake, the amount of additional contamination that may be permitted in various local cases without endangering the contiguous water supplies and the best methods of disposing of the sewage from each individual community so all possibility of contamination may eventually be eliminated.

The concrete, economical value to a community of a safe and satisfactory supply of water, aside from the great desirability, from a humanitarian standpoint, of obtaining such a supply, fully justifies any financial expense that may be involved in eradicating the present highly unsanitary conditions existing adjacent to the water intakes of certain cities along Lake Michigan. The most remarkable local instance of this economic value is a reduction in the typhoid fever death rate in Chicago of 89.8 per cent. from 1891 to 1907, due chiefly to the opening of the Drainage Canal. This evident economic value of the results to return from the campaign started by the new organization, the existing community of interest among the municipalities and states concerned and the certainty of active participation in the movement by capable men occupying positions enabling them to secure results, will doubtless culminate in legislation to keep the sewage and wastes of the towns and cities along the shores of the lake out of the sources of domestic water supply. This legislation will doubtless be based on thorough study of the existing conditions and of the methods of removing the sources of contamination that are best suited to each locality, such as the commission appointed evidently intends to undertake. The eventual enforcement of wise legislation along these lines will unquestionably greatly improve the quality of the waters of Lake Michigan and make them fit for use by the people of the great, rapidly growing centers of population on the shores of that lake.

Notes and Comments.

The Hudson Bay Railway project received another slight impetus a few days ago when the Canadian House of Commons resolved by a unanimous vote that the time had arrived for the construction of a railroad to Fort Churchill, on the west coast of that inland sea. Resolutions do not amount to much, as a rule, but this one is interesting because it shows that the Canadian authorities have not dismissed the project of building this road. Two years ago a large amount of testimony was taken by a committee of the Canadian Senate, which indicated that possibly 10,000 square miles of excellent farming land, several times as much land of fair fertility and perhaps 100,000 square miles of pulp timber would be opened up by this road. Moreover, the fisheries of Hudson Bay would furnish some business for the line, which might develop into a large trade. The Canadian Northern Railway now runs to a point on the Saskatchewan River only 450 miles by trail from Fort Churchill, and from this river fish is now shipped to Chicago. This journal has been informed by people acquainted with the district west of Hudson Bay that the territory is one capable of supporting a large population, if the settlers are thrifty and competent, and the reason assigned for its backward development is the reluctance of the Hudson Bay Company to have the territory change its character. Be that as it may, a railway will doubtless be built before long into the heart of Keewatin, for this great tract of territory west of the Bay is now the only part of the Dominion of known natural richness where a railroad does not run or is not under construction.

The City of Philadelphia needs a great many improvements, according to the annual message of Mayor Reyburn, some of them to make it more beautiful, others to provide better means of communication within its boundary and out into its suburbs, and others to attract a larger share of water-borne commerce than now reaches its wharves. His Honor is very frank in his criticisms of shipping facilities at the City of Brotherly Love, and his statements are approved in the local papers. Its railroad facilities have been kept so well up to the demands made on them that there would be some excuse for a failure to appreciate the backward condition of the waterfront, but that it has been recognized is shown by the hearty response to the proposal to straighten the river channel, build bulkheads around the whole waterfront with wide "farms" or commercial streets behind them, and construct a belt railroad around the line of bulkheads. Such a project is an enormous one, but it deserves the most thoughtful consideration. Now that the Delaware has been deepened, the city's standing among the ports of the world can be advanced far ahead of its present position if the city itself will act with the same shrewdness that has always characterized its merchants in their private dealings. A vessel is earning nothing while it lies at a wharf, and those ports in which cargoes are unloaded and loaded most quickly will be given the preference by shipowners, other things being equal. A system of commodious wharves properly supplied with mechanical equipment and connected by a belt line with the railroads entering the city should prove directly profitable and indirectly invaluable to Philadelphia.

Another Wharf Project has been advocated within a few days on the other side of the continent, for the purpose of attracting to Oakland a larger share of the shipping that enters San Francisco Bay. The San Francisco fire and the advent of new railroads have made changes in commercial conditions within the Golden Gate, and the people of Oakland believe that the time has come when the advantages of their city as a seaport should be utilized. For many years transportation affairs in San Francisco were directed by a single great master-mind, but that condition has changed. His railroad is no longer the sole one to reach the bay and the steamships of the lines he managed are slowly having their trade taken from them by heavily subsidized vessels flying the Mikado's flag. The Japanese ships which are slowly acquiring control of the Pacific trade and the vessels of other nations which are engaged in traffic with Europe are more independent than those of earlier days, and in this condition Oakland sees an opportunity for advancement. It labors under a great physical disadvantage, however, arising from the enormous amount of silt, brought down by the Sacramento River, which has formed wide flats along the waterfront of the city making any ordinary methods of harbor improvement impracticable. Training walls and land reclamation must probably form part of any comprehensive system of docks, but it needs only a glance at the map to see that if these works can be constructed without too great expense they should eventually prove a source of profit to the city. A system of wharves and warehouses connected with the railroads reaching Oakland presents commercial possibilities of great attractiveness. The Chamber of Commerce of the city has brought the subject to the attention of the local authorities, and, while no immediate action is likely, it would seem that steps might well be taken to have the situation investigated by engineers familiar with harbor improvements. The city is in danger of having its waterfront pass into the control of railroad companies, and experience indicates that this condition is undesirable. While every reasonable facility should be given to such companies for the transaction of their business, a city which allows the control of its waterfront to pass into private hands is making a grave mistake.

Dams Across Navigable Streams have aroused a sharp issue in Washington between the President and some of the Western congressmen. The Senate recently passed a bill granting to a private company the privilege of damming the Snake River in the State of Washington. This measure has been opposed by senators from Idaho on the ground that the dam would seriously interfere with the navigability of the stream in that State. It is reported that the President will veto this bill and that the issue will thus be brought to the attention of the public. It is perfectly well known that the President is deeply opposed to granting privileges involving water rights unless some return is made to the Government for them. Wherever such applications for privileges have been referred to the Corps of Engineers of the army for opinions, a favorable report on any of these requests has been accompanied by a recommendation that the company obtaining the privilege be required to pay a proper compensation for it. Just how to measure the compensation is a difficult matter in a good many cases, but it is safe to say that the President is upheld by public opinion in his position that some compensation, immediate or future, should be paid. The subject was brought up in another form on April 13, when the President vetoed a bill giving the Rainy River Improvement Co. three years longer in which to build a dam on Rainy River. "I do not believe," the President wrote, "that natural resources should be granted and held in an undeveloped condition either for speculative or other reasons."

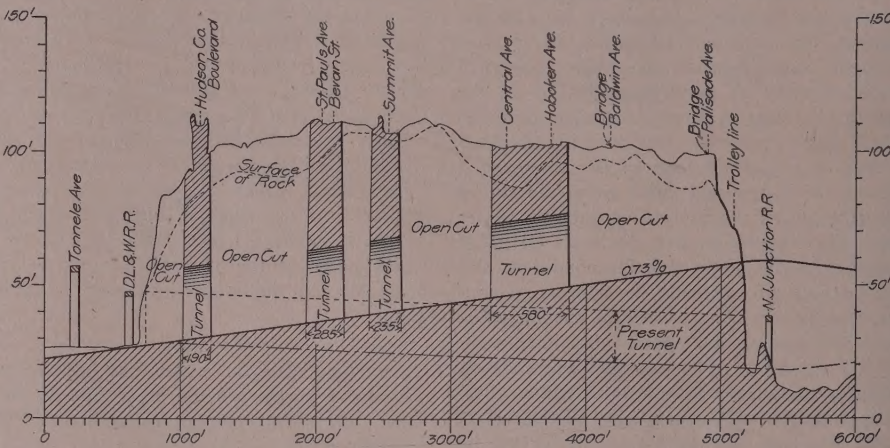
Culvert Pipe of cast-iron and reinforced concrete has been tested at the laboratory of applied mechanics of the University of Illinois, and at the meeting of the Western Society of Engineers on Wednesday of this week Prof. A. N. Talbot read a paper on the results of these experiments. The first portion of the paper gives a theoretical analysis of the stresses existing in such culverts under both concentrated and distributed loads. In the second portion of the paper the tests made with nine cast-iron and five reinforced concrete culvert pipes and rings of both materials are reported at some length and the results are compared with those of the theoretical analysis. There is a fairly close agreement between the tests made with concentrated loads and the theoretical results, but the tests made with distributed loads show far less agreement. This confirms the general opinion among engineers who have had occasion to examine the condition of culverts under embankments of different classes, for experience has shown that the bedding of the bottoms of the pipes and the backfilling about and over them exercise a marked influence on their freedom from cracks. If the pipe is bedded in loose material, the effect of settlement will be to compress the earth immediately under the bottom of the pipes more completely than will be the effect at one side, with the result that the pressure will not be uniformly distributed horizontally. Similarly, if the material at the extremity of the horizontal diameter is loose and offers little restraint, the pressure on the earth will not be distributed horizontally and the amount of bending moment will be materially different from that where careful bedding and tamping give an even distribution of the bearing pressure over the bottom of the sewer. Where there is any question of strength, the paper shows that it will be money well spent to take care in bedding the pipe and in backfilling around and over it. A little extra expense in such work will add considerable stability, life, strength and safety to such structures, far out of proportion to the additional cost.

THE NEW FOUR-TRACK ENTRANCE OF THE ERIE RAILROAD INTO JERSEY CITY.

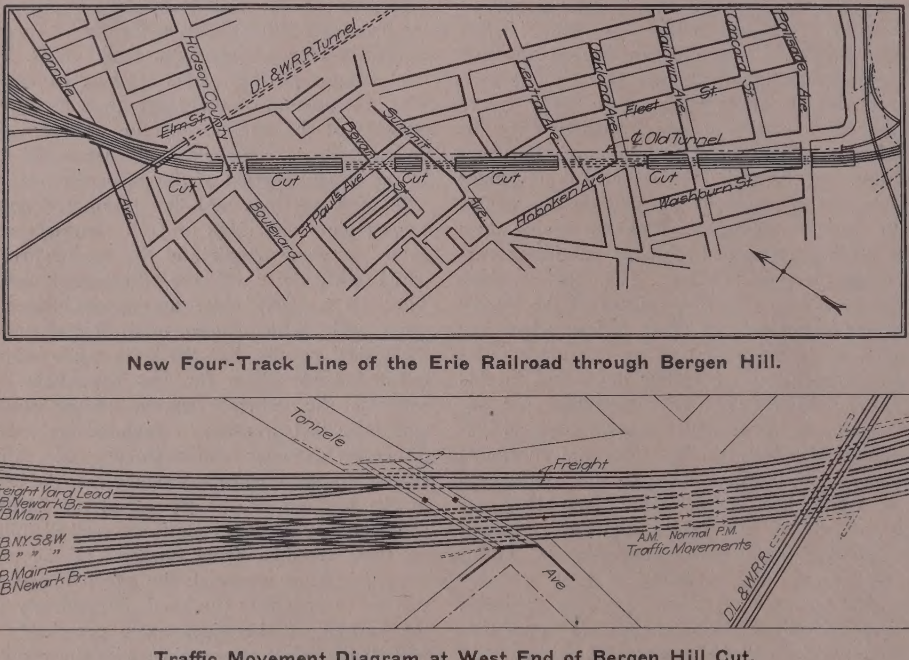
The Erie Railroad now has under construction a new 4-track line through Bergen Hill, a continuation of the Palisades, which, at the point where the Erie Railroad strikes it, is about three-fourths of a mile from the bank of the Hudson River. All of the roads entering New York from the West are obliged to contend with this barrier, except the Central R. R. of New Jersey, which reaches its terminal on the Hudson River by a trestle across Newark Bay, considerably south of its station. The Pennsylvania Railroad comes through the hill in an open cut, and the Erie, Delaware, Lackawanna & Western and West Shore roads through tunnels. The ridge, immediately west of the Erie depot, has a height of about 100 ft., and is about 4,000 ft. wide.

In addition to a large through traffic the Erie has a very heavy suburban business, which separates immediately to the west of Bergen Hill to the various New Jersey towns. The remarkable growth in traffic has for a number of years caused much congestion at the terminal and betterments have been under consideration for a long time. The plans for adequately handling the traffic call for a new terminal eventually, in addition to the new 4-track entrance through the hill, which is described in this article.

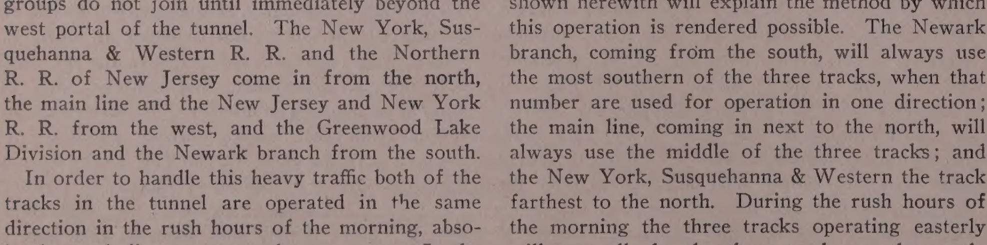
At the present time the business is handled entirely by means of a two-track tunnel, its operation being reserved for passenger trains in the rush hours of the morning and evening, while both passenger and freight business is handled during the rest of the day. The traffic of the Erie originates on six different branch lines, which merge into three groups, of two lines each, some distance west of Jersey City. These three groups do not join until immediately beyond the west portal of the tunnel. The New York, Susquehanna & Western R. R. and the Northern R. R. of New Jersey come in from the north, the main line and the New Jersey and New York R. R. from the west, and the Greenwood Lake Division and the Newark branch from the south.



Profile of the Four-Track Line through Bergen Hill.



New Four-Track Line of the Erie Railroad through Bergen Hill.



Traffic Movement Diagram at West End of Bergen Hill Cut.

In order to handle this heavy traffic both of the tracks in the tunnel are operated in the same direction in the rush hours of the morning, absolutely precluding any westerly movement. In the evening, however, instead of confining the operation of the tunnel tracks to trains of but one direction, the trains of each group are double-headed, the two locomotives which pull the joint train cutting off where lines of the group diverge and picking up their respective parts of the train.

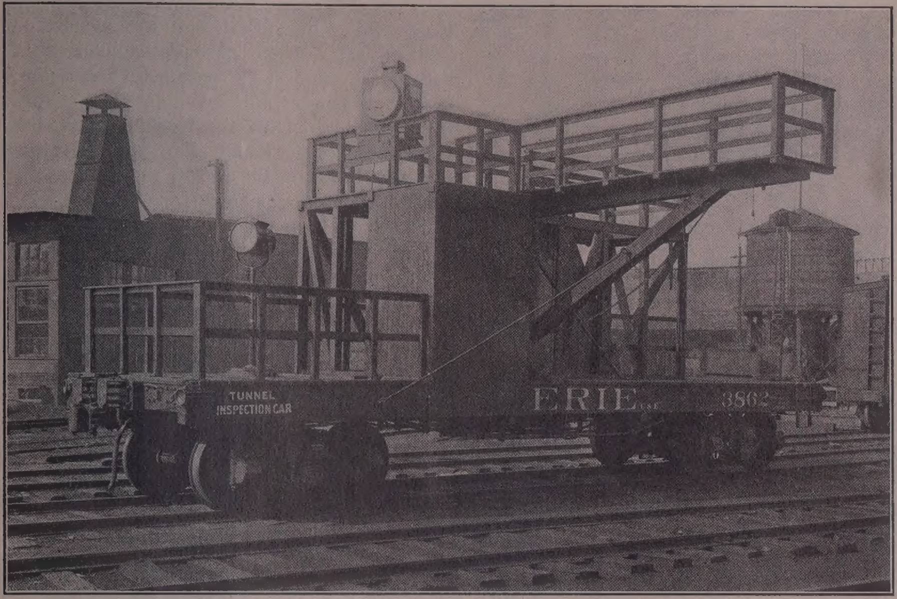
The plans now being carried out will provide a four-track entry devoted entirely to passenger business, the freight traffic being handled through the present tunnel. It is planned to operate two of the tracks in each direction during the hours of normal traffic, and three of the tracks in one direction and a fourth in the opposite direction during rush hours. This will enable three trains to enter or leave the terminal abreast of each other at the same identical time, and by a further system of improvements on the west side of Bergen Hill, to proceed on their respective lines without interfering with each other or with the opposite-bound trains using the remaining one of the four tracks. A reference to the diagram shown herewith will explain the method by which this operation is rendered possible. The Newark branch, coming from the south, will always use the most southern of the three tracks, when that number are used for operation in one direction; the main line, coming in next to the north, will always use the middle of the three tracks; and the New York, Susquehanna & Western the track farthest to the north. During the rush hours of the morning the three tracks operating easterly will naturally be the three tracks on the south, while at evening, for the westerly movement, the three tracks to the north will be used.

This method of operating the tracks of the four-track section is apparently but part of the solution of the problem, as it takes no account of the crossings of the tracks. The latter is provided in a system of separated crossings, by means of which none of the six tracks of the three double-track groups intersects at the same level. These separated crossings, or flying junctions, as they are called, are located shortly to the west of the hill and will involve the construction of a number of miles of new embankments besides the necessary masonry and bridge work at the points of intersection.

The four tracks emerging from the west end of the cut pass beneath the Delaware, Lackawanna & Western R. R., and under Tonnele Ave., and then diverge into six tracks, two for each of the operating groups above described. One of each pair of tracks will always operate in-bound, or eastward, and the other out-bound or westward. At the point of diversion from four to six tracks, a set of crossovers will be installed so that all inbound tracks can be connected to three tracks in the cut and all outbound tracks to the fourth track, these combinations being possible also when the three tracks are operating outbound. One set-up of the switches will provide for all movements inbound in the morning and outbound in the evening, the operator thereafter being concerned only with diverting the light traffic of the contrary movements to or from the proper lines.

The westbound Newark branch and Main Line tracks are those to the north, and swing outward to pass beneath the freight and both passenger tracks of the New York, Susquehanna & Western R. R., meanwhile diverging from each other. The object of this diverging is to permit the Main Line tracks to rise on 1.04 per cent. grade in order to pass over the Newark branch, the latter continuing on a down-grade of 0.55 per cent. The passenger tracks of the New York, Susquehanna & Western R. R. parallel each other, being the central ones of the group of six tracks. From the point of diversion just west of Tonnele Ave., these lines rise, going west, on a 1.15 per cent. grade, swing parallel to the freight tracks of the same line, and cross above the westbound tracks of the Newark branch and the Main Line. The two remaining tracks to be accounted for, namely, the eastbound tracks of the Main Line and of the Newark branch, are the two southernmost of the group of six. That of the Main Line crosses over the westbound Newark Branch and then runs parallel to the eastbound tracks of the latter, passing side by side under the freight tracks of the New York, Susquehanna & Western R. R., the eastbound main dropping down from its overcrossing above the Newark branch to its undercrossing beneath the freight line mentioned on a 1.07 grade, while the eastbound Newark branch rises continuously on a 0.36 per cent. grade.

A study of the accompanying plan of the new layout will show the magnitude of the undertaking, the great amount of embankment to be thrown up and the piers and abutments for the bridges at the separated crossings. The scheme as herein described requires the construction of 8.4 miles of new double-track and the abandonment of 8.7 miles of double-track, which latter can in most cases be used for freight business. The engine facilities, lying directly west of Tonnele Ave., will be transferred to a new location and large changes will have to be made in the trackage of the east end of the freight yard. Connected with the abandoning of the Newark branch tracks is the construction on the new line of a double-track truss drawbridge, having a girder approach with spans 449 ft. in length, which with the draw-span of 339 ft. gives a distance between abutments of 788 ft. The new embankment will involve about 530,000 cu. yd. of material, part of which will come from the cut and part from borrow pits.

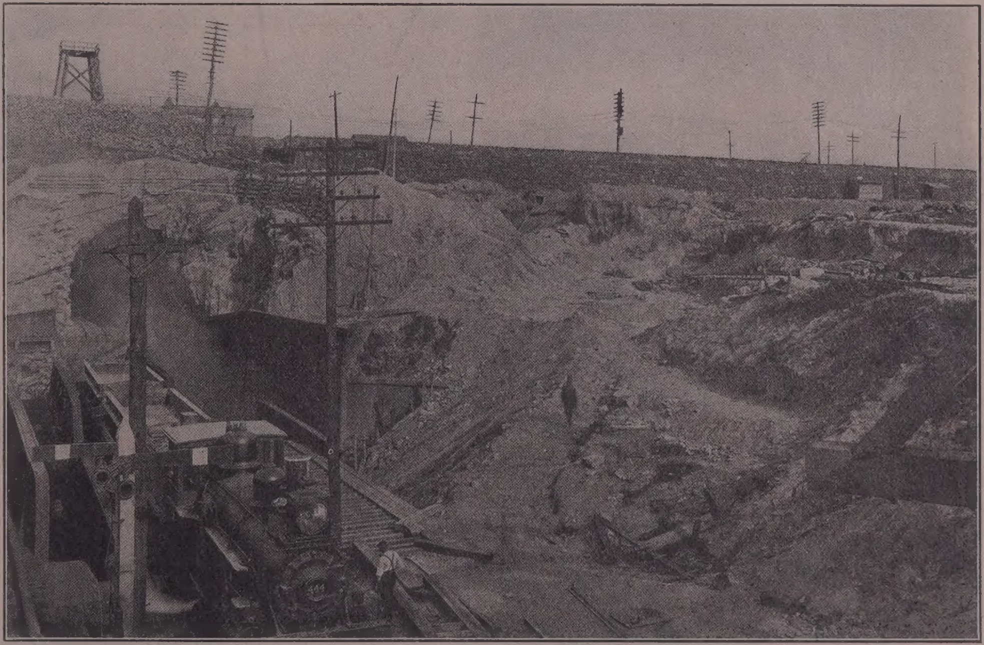


Car Used for Inspecting Present Tunnel after Blasting in Cut.

The track arrangement just described not only prevents all interference of the passenger trains with each other, whether operating in the same or in opposite directions, but also separates the passenger from the freight business, so that there can be no possible interference of these two within four miles of the terminal in Jersey City.

The capacity afforded by the new facilities is more than sufficient for the present needs of the system and will provide for a large growth in traffic without overtaxing. The improvements, as will be noted in the paragraphs above, will allow the New York, Susquehanna & Western, which is controlled by the Erie Railroad, to use the Erie terminal, although the line at the present time runs its trains into the Pennsylvania Station in Jersey City.

Although the present line of the Erie through the hill is carried in a tunnel, as is the Delaware, Lackawanna & Western R. R., which is now completing an additional two-track tunnel, the four-track system is being built in an open cut, with just four short tunnels, the longest 580 ft. long, to carry certain city streets. There were a number of reasons for adopting an open cut rather than a tunnel, not the least of which was the greater desirability of an open cut from the standpoints of operation and agreeableness to passengers. Not only are there considerable difficulties with a tunnel wide enough to accommodate four tracks, but the cost of tunneling operations is necessarily higher than for open cut work. Another large factor is the great value which crushed rock of the high quality of the trap of the Palisades has in the vicinity of New York City for ballast and concrete. The salvage of the rock represents a considerable item, which offsets the higher cost of the removal of so much material.



Beginning Bergen Hill Cut, Showing Portals of Present Erie and New Lackawanna Tunnels.

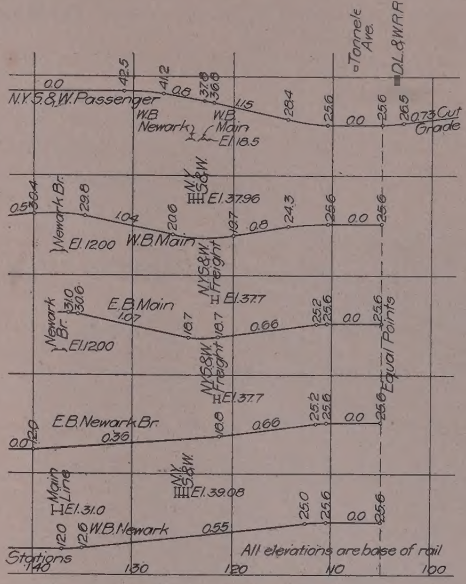
The open cut will have the same length as the present tunnel, about 4,450 ft. It will vary in depth from 40 to 75 ft., has a width at the bottom of 60 ft., and a maximum width at the top of 100 ft. The four tunnels, the shortest 190 ft. long and the longest 580 ft., will be four-track sections, lined with concrete. Their location with reference to the open cut is shown on the accompanying plan. The open cut excavation will involve the removal of about 420,000 cu. yd. of rock and 114,000 cu. yd. of earth, while the tunnel excavation will amount to about 79,000 cu. yd., all of which is rock.

The new cut lies parallel to, and with its center line 67 ft. south of, the present Erie tunnel. The grade of the latter rises going west, while the grade of the open cut is in the opposite direction, having a gradient of 0.73 per cent., which was fixed by certain limiting conditions at both ends of the cut. At the western end the Erie now passes under the Delaware, Lackawanna & Western R. R., right at the portal of the tunnel of the latter road, and the new lines were planned for a similar arrangement, in order to keep the four-track system on the same level at this end with the present two-track tunnel line. At the eastern end of the cut the line crosses that of the New Jersey Junction R. R. and the alternative presented itself of crossing under the latter, as does the present Erie line, or going over it. The latter of the two schemes has been adopted. This not only lessens by a considerable amount the quantity of excavation, but allows the freight business of the road, which will be handled through the present tunnel, to be separated to both sides of the passenger terminal without interfering in any way with the passenger business, this being particularly important, as the Erie operates freight yards on both sides of its passenger station on the Hudson River water front. The difference in elevation between the rails in the open cut and in the present tunnel at the eastern end will be about 40 ft., and in order to reach the same level at the western end the gradient of 0.73 per cent. is necessary. The line is crossed by a number of streets of Jersey City Heights, one of the resident sections of the city, and it was first planned to carry these over the cut on bridges. Further consideration, however, resulted in the determination to build tunnel sections in the cut to carry these streets rather than adopt the other plan proposed. This has the advantage of allowing the street traffic to remain undisturbed and avoids the removal of sewerage, gas, and water mains. Two streets, however, will be carried over the cut on bridges, Baldwin Ave. on a reinforced concrete structure and Palisades Ave. on a steel bridge. At the eastern end there will also be a bridge across the cut to carry the tracks of a trolley line.

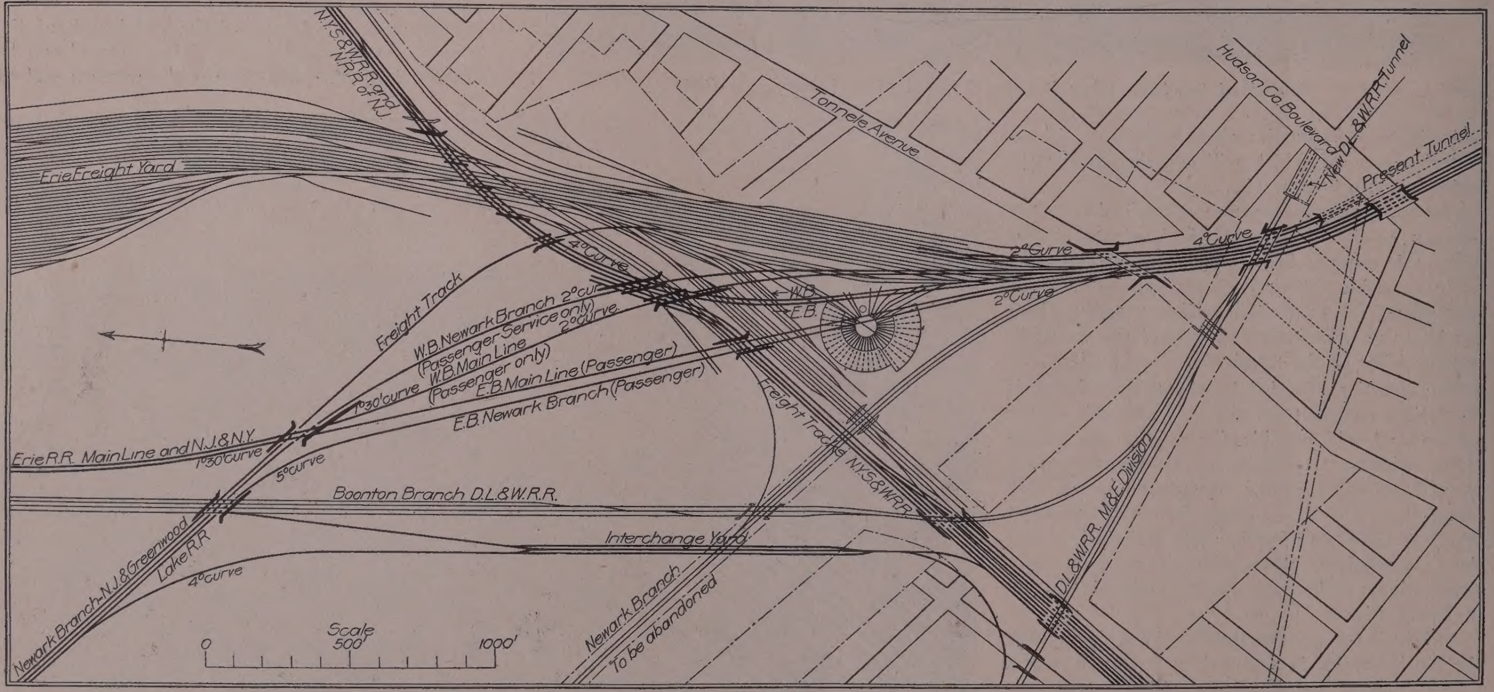
The contract for the work is held by the Millard Construction Co., of Philadelphia, who have placed a considerable plant on the ground, for the excavation, removal, storage and distribution of the excavated materials. A 3-ft. gauge track has been laid the full length of the cut so that all of the material excavated can be hauled in 4-yd. dump cars to the western end of the work, where the top soil is used for embankment upon the Hackensack Meadows and the rock carried to a large crushing plant. A temporary tunnel for the handling of excavated material has been driven beneath the Hudson County Boulevard, and through it is hauled all of the material taken out in the prosecution of the work. About 125 cars and 8 Porter locomotives are used on the job, a number of balanced cable cranes and a great many movable derricks in the bottom of the cut and stationary ones at the top. Four balanced cable cranes are used to span open cut No. 4, transversely to the center line, while another one is placed over open cut No. 3 parallel with the center line, the latter cut being quite short. The cranes have a span of 240 ft., and a rated capacity for lifting 5 tons. In the excavation of open cuts Nos. 1, 2 and 3, derricks, handling scale pans, will be used exclusively, movable derricks being placed in the bottom of the cut and stationary ones at the top for hoisting out the loaded pans, and dumping them into the narrow-gauge spoil cars.

The work is being prosecuted most actively at the portals of the tunnels so as to reach their level as soon as possible and allow the starting of the headings, thus permitting the use of a larger force and driving the work at a greater rate. The rock encountered is a very hard trap, considerably fissured in places. It is being taken off in 12-ft. lifts, the blasting operations requiring the constant use of 60 Sullivan drills and 12 hand drills for blocking.

On account of the work being prosecuted in a populated neighborhood and the fact that it lies close to the existing Erie tunnel, progress is necessarily slow and beset with difficulties which would be absent were the work in an isolated place. Blasting is restricted to such times as there is a lull in the traffic movement through the tunnel. There are three such periods in the day when passenger travel is light, during which, if blasting be desirable, the freight traffic through the tunnel is for the time suspended. Extraordinary precautions are taken to safeguard operation through the tunnel and for this purpose, in addition to the very excellent safety devices constantly employed in it, a special blasting operator has been stationed in an office alongside of the cut, near its center, at the top of the hill. This operator has connections with both ends of the tunnel and from him directly the blasting foreman secures permission to blast. When shots are ready at the blasting periods the blasting operator informs the superintendent of the tunnel, who has absolute control of train movements at both portals, that traffic must be suspended during the period which he specifies. Accompanying this information is a statement of the number of blasts to be fired, their exact location and the time the shots are to be fired. When the tunnel superintendent receives these orders he, in turn, reports when operation through the tunnel has been suspended, and not until then does the blasting operator give written permission to the contractor to fire his shots. This written permission is signed by the blasting operator, for the railroad, and the blasting fireman, for the contracting company, and a copy of it is furnished to the tunnel inspector. As soon as the blasting foreman has received permission to fire, he throws on a series of switches which light signal lamps at each cut, so as to avoid all delay in firing the charges.



Profiles of Lines at Flying Junctions.


N.Y.S.&W.R.R. and N.R.R. of N.J.
Erie Freight Yard
Tonnele Avenue
Hudson Co. Boulevard
New D.L.&W.R.R. Tunnel
Present Tunnel
2° Curve
4° Curve
Freight Track
W.B. Newark Branch 2° curve (Passenger Service only)
W.B. Main Line 2° curve (Passenger only)
E.B. Main Line (Passenger)
E.B. Newark Branch (Passenger)
1°30' curve
W.B.
E.B.
Erie R.R. Main Line and N.J.&N.Y.
1°30' curve
5° curve
Boonton Branch D.L.&W.R.R.
Freight Track
N.Y.S.&W.R.R.
Interchange Yard
D.L.&W.R.R. M.&E. Division
Newark Branch—N.J.&Greenwood Lake R.R.
4° curve
Newark Branch—To be abandoned
Scale
0
500'
1000'

Plan of Flying Junctions at Entrance to the Four-Track Line of the Erie Railroad through Bergen Hill.

When the shots have been fired, an inspection car, shown in an accompanying cut, is run into the tunnel and the arch and side walls carefully examined to discover whether the blasting has displaced or loosened the lining or rock in the tunnel. The car is fitted with two powerful acetylene projection lamps, while a swinging platform can be opened so as to extend across the track adjacent to the one upon which the car is standing. Upon the completion of the inspection and a report that all is safe, the tunnel operation is resumed.

The power house for supplying compressed air for the drills, the crusher plant and the repair shop are located at the west end of the cut. The crushing plant contains one No. 10 and four No. 6 McCully gyratory crushers. The No. 10 crusher takes the rock just as it comes from the cut, and discharges the crushed material into a 30-in. bucket conveyor, which elevates it to two 14-ft. by 5-ft. 6-in. screens of 2½-in. mesh. The material rejected from these screens passes through chutes to the No. 6 crushers and so continues round until all of it is of 2½-in. size. It then passes to a 410-ft. conveyor, which rises to a height of about 75 ft., to the top of a distributing tower. When as much stone has been deposited about the latter as is possible, a traveling stacker will be provided, which will have an ultimate travel of 700 ft. This stacker will carry a 62-ft. boom, upon which runs a belt conveyor, so that the material can be deposited in front of and to both sides of the stacker. Between the distributing tower and the stacker another belt conveyor of adjustable length will be installed, the slack being taken up by a counterweight. As the stacker moves out, new belts of greater length with proper supports will have to be provided. The engine driving the crushing plant, including the five crushers and all of the belt conveyors except those on the top of the storage pile, is a 500-h.-p. cross-compound operating at 125 r.p.m. The dump cars which deliver stone to the No. 10 crusher are tipped with the aid of an air cylinder and a small hand-operated derrick, installed alongside of the large crusher, enables rock jams to be easily broken and allows rocks that are too large to be removed.

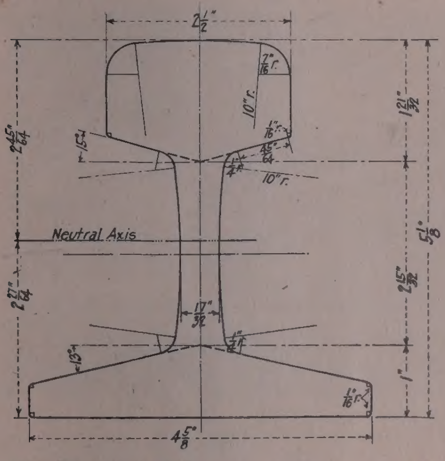
Three Laidlaw-Dunn-Gordon cross-compound air compressors are installed, the capacity being sufficient for about 70 drills. The air supply is carried along the cut through a riveted pipe line. The boiler room contains four 500-h.-p. water tube boilers. A Sturtevant fan supplies forced draft and the feed water is heated in a heater furnished by the Platt Iron Works. A small repair shop for making repairs to cars and locomotives occupies a bay adjacent to the boilers. The power and crusher plants were supplied by the Power & Mining Machinery Co., Cudahy, Wis.

The work is being carried on under Mr. J. M. Graham, vice-president of the Erie Railroad, in charge of engineering; Mr. F. L. Stuart, chief engineer, and Mr. Geo. H. Burgess, engineer of terminal improvements. The resident engineer on the work is Mr. A. L. Moorshead. Mr. John Hendrie is the superintendent for the Millard Construction Co.

Cost of Sand and Gravel For Gunnison Tunnel Lining.

According to notes prepared by Mr. F. W. Hanna, engineer, U. S. Reclamation Service, from a report by Mr. E. L. Driggs, junior engineer, U. S. Reclamation Service, stationed at Montrose, Colo., large quantities of sand and gravel are being used in connection with the construction of the Gunnison Tunnel by the Reclamation Service near Montrose. The sand and gravel are being used for aggregates in the tunnel concrete lining, that for the West Portal being obtained from a gravel pit located on top of a hill nearby. The sand and gravel suited for use without crushing or washing are found in a natural deposit overlaid with a very thin covering of soil. In the development of the gravel bed the soil covering was removed by means of scrapers and teams. The gravel is then loaded by hand into small cars, running on portable tracks. These cars are pushed to the edge of the hill and the contents there dumped into a chute about 200 ft. long leading to bins at the foot of the hill. This chute is provided with a system of screens so arranged as to remove all of the largest boulders and to separate the gravel and sand.

The separated gravel and sand are loaded from the bins into dump wagons having a capacity of 2 cu. yd. each and hauled with teams about one-fourth of a mile to the main shaft of the tunnel where they are dumped directly into bins in the shaft. From this point the aggregates are fed directly into the concrete mixer located within the tunnel at the foot of the shaft.



Area, Head, 3.57 sq. in.	42.2 per cent
Area. Web, 1.51 sq. in.	17.8 per cent
Area, Base, 3.39 sq. in.	40.0 per cent
Total Area, 8.47 sq. in.	100.0 per cent
Moment of Inertia	29.1
Section Modulus Head	10.77
Section Modulus Base	12.02
Ratio Periphery Head to Area Head	1.73
Ratio Periphery Web to Area Web	3.81
Ratio Periphery Base to Area Base	2.60
Ratio Total Periphery to Total Area	2.48

Section of 85-Pound Rail.

Careful records have been kept of the cost of the sand and gravel used in the tunnel, and data covering the output for five months during 1907 have been here brought together. During this period there were hauled a total of 9,112 cu. yd., employing for the whole sand and gravel plant operation 1 foreman, 9 laborers and 3 teamsters. The foreman received $3.04 per 8-hour day, the laborers $2.56, and the teamsters $2.40. All equipment is owned by the United States, and the item of miscellaneous cost covers horse feed and care, repairs to harness, cars and wagons, and the cost of all shovels and other small tools.

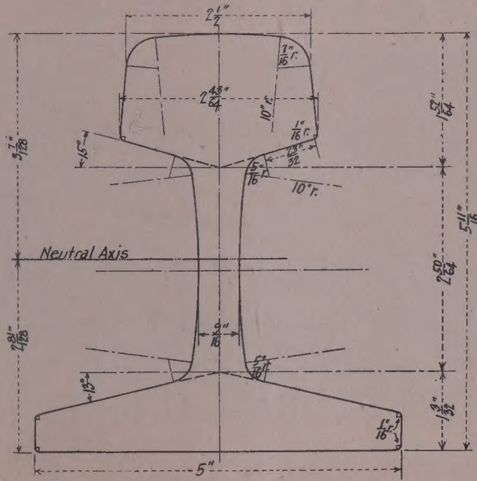
COST OF SAND AND GRAVEL FOR GUNNISON TUNNEL LINING.

Distribution of Cost.	Total Cost.	Unit Cost
Foreman	$445.74	$0.049
Laborers	3,539.52	0.388
Teamsters	839.80	0.092
Miscellaneous	938.81	0.108
Totals	$5,763.87	$0.637

The New Steel Rail Specifications of the Pennsylvania System.

In view of the increasing severity of road service and the recognized necessity for developing an improved steel rail, the Pennsylvania Railroad has for some months been conducting an exhaustive examination of the entire art and practice of rail manufacture. The inquiry has been of an exceptionally thorough character and, though important results have been attained, the work of investigation will be continued. General conditions have made it desirable to develop, if possible, a rail which should reduce the expense of maintenance and provide increased resistance against the growing requirements of road service. An incidental purpose of the plan was to secure uniformity in the rail practice of the lines east and west of Pittsburg. The series of experiments undertaken—though probably unprecedented in their completeness and in the importance of the scientific data which they are supplying—represent merely a continuance of the Pennsylvania's policy to conduct tests and inquire at all times as to the possibility of improving its present practice in any direction of activity.

To conduct this inquiry, a large special committee—composed not only of officers of the railroad, but of two of the important steel manufacturers—was appointed by the Pennsylvania System in June, 1907. The membership of that committee was as follows: Theo. N. Ely, chief of motive power, lines east and west, chairman; A. C. Shand, chief engineer, lines east; L. R. Zollinger, engineer maintenance of way, lines east; A. W. Gibbs, general superintendent motive power, lines east; C. B. Dudley, chemist, lines east; T. H. Johnson, consulting engineer, lines west; Robt. Trimble, chief engineer maintenance of way, lines west; W. C. Cushing, chief engineer maintenance of way, lines west; J. C. Bland, engineer of bridges, lines west; D. F. Crawford, general superintendent motive power, lines west.



Area, Head, 4.09 sq. in.	41.0 per cent
Area, Web, 1.85 sq. in.	18.6 per cent
Area, Base, 4.03 sq. in.	40.4 per cent
Total Area, 9.97 sq. in.	100.0 per cent
Moment of Inertia	41.9
Section Modulus Head	13.71
Section Modulus Base	15.91
Ratio Periphery Head to Area Head	1.59
Ratio Periphery Web to Area Web	3.58
Ratio Periphery Base to Area Base	2,43
Ratio Total Periphery to Total Area	2.30

Section of 100-Pound Rail.

The investigations and discussions of this committee have now resulted in the development of rail sections and specifications which are believed to be a distinct improvement upon those hitherto in use on the Pennsylvania System. To make a practical test of the work of the committee, the company placed, in December, 1907, an experimental order for 10,000 tons of rails—a part of the 1907 allotment—to be rolled in accordance with the new sections and specifications. The experience with these orders has resulted in some slight changes in the specifications, but no change has been made in the sections as originally adopted. The Pennsylvania System has adopted as a standard for its 1908 orders the new sections and the specifications as revised Feb. 4, 1908. These specifications and sections are now made public, and are as follows:

"*Chemical Composition.*—1. The steel of which the rails are rolled shall conform to the following limits in chemical composition:

	Lower limit.	Desired composition.	Upper limit.
Bessemer steel rails:			
Carbon	0.45%	0.50%	0.55%
Manganese	0.80	1.00	1.20
Silicon	0.05	0.12	0.20
Phosphorous			0.10
Open hearth steel rails:			
Carbon	0.70%	0.75%	0.80%
Silicon	0.05	0.12	0.20
Manganese			0.80
Phosphorous			0.03

"*Process of Manufacture.*—2. Ingots shall be kept in a vertical position until ready to be rolled, or until the metal in the interior has had time to solidify.

"3. No 'bled' ingots shall be used. ('Bled ingot'—One from the center of which the liquid steel has been permitted to escape.)

"4. There shall be sheared from the end of the bloom formed from the top of the ingot sufficient discard to insure sound rails. (All metal from the top of the ingot, whether cut from bloom or rail, is the top discard.)

"5. In reheating, care shall be taken to avoid burning the steel, and under no circumstances shall a 'cinder heat' be used. ('Cinder heat'—One in which the scale on the sides of the ingot becomes fluid.)

"6. The number of passes and speed of train shall be so regulated that on leaving the rolls at the final pass the temperature of the rail will not exceed that which requires a shrinkage allowance at the hot saws, for a rail 33 ft. in length, of 6¼ in. for 85-lb. section, and 6½ in. for 100-lb. section. These allowances to be decreased at the rate of 0.01 in. for each second of time elapsed between the rail leaving the finished rolls and being sawed. The bars shall not be held for the purpose of reducing their temperature, nor shall any artificial means of cooling them be used between the leading and finishing passes, nor after they leave the finishing pass.

"*Mechanical Requirements.*—7. The name of the maker, the weight and type of the rail, and the month and year of manufacture shall be rolled in raised letters and figures on the side of the web, and the number of the heat shall be plainly stamped on each rail where it will not subsequently be covered by the splice bars. A letter shall be stamped on the web to indicate the portion of the ingot from which the rail was rolled.

"8. The section of rail shall conform as accurately as possible to the templet furnished by the railroad company. A variation in height of 1/64 in. less or 1/32 in. greater than the specified height, and 1/16 in. in width of flange, will be permitted; but no variations shall be allowed in the dimensions affecting the fit of the splice bars.

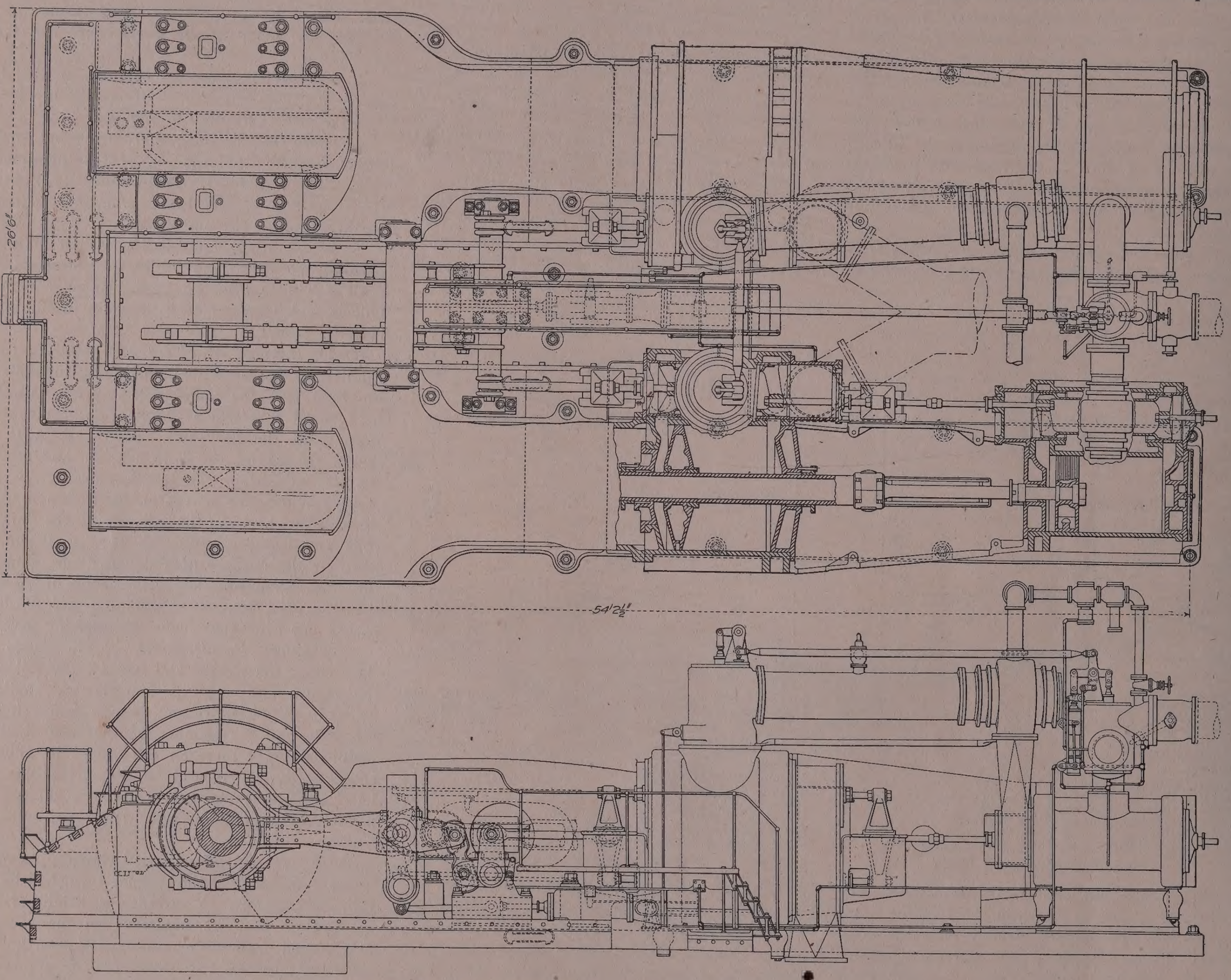
"9. The weight of the rails specified in the order shall be maintained as nearly as possible, after complying with the preceding paragraph. A variation of ½ of 1 per cent. from the calculated weight of section, as applied to an entire order, will be allowed.

"10. The standard length of rails, at a temperature of 60° Fahr., shall be 33 ft. Ten per cent. of the entire order will be accepted in shorter lengths, varying as follows: 30 ft., 27½ ft., and 25 ft.; and all No. 1 rails less than 33 ft. long shall be painted green on ends. A variation of ¼ in. in length from that specified will be allowed.

"11. The rails must be free from injurious mechanical defects and flaws; shall be sawed square at the ends, a variation of not more than 1/32 in. being allowed; and burrs shall be carefully removed.

"12. The rails shall be smooth on the heads, straight in line and surface, and without any twists, waves, or kinks; particular attention being given to having the ends without kinks or drop. The hot straightening shall be carefully done, so that gagging under the cold presses will be reduced to a minimum. Any rail coming to the straightening presses showing any sharp kinks or greater camber than that indicated by a middle ordinate of 4 in. in 33 ft. will be at once marked as a No. 2 rail, and only accepted as such. The distance between the supports of rails in the straightening presses shall not be less than 42 in.

"13. Circular holes for splice bars shall be drilled to conform accurately in every respect to the drawing and dimensions furnished by the railroad company, and must be free from burrs.



General Features of the Allis-Chalmers Engine for the South Sharon Rolling Mills.

"*Tests and Inspection.*—14. One drop test shall be made on a piece of rail not less than 4 ft. and not more than 6 ft. long, selected by the inspector from each heat of steel, from any position in the ingot. The test piece shall be placed head upwards on solid supports, 5 in. radius, 3 ft. between centers, and both 85-lb. and 100-lb. sections shall be subjected to an impact test from a weight of 2,000 lb., falling 15 ft. The deflection for No. 1 classification rails must not exceed that indicated by a middle ordinate of 2 in. in 3 ft. for 100-lb. section, and 2¼ in. for 85-lb. section. Test pieces shall be tested to destruction. The temperature of the test pieces shall be between 60° and 120° Fahr.

"(*a*) If a test piece breaks without showing physical defect, all rails made from that heat shall be rejected absolutely. (*b*) If, however, the test piece broken under test ('*a*') shows physical defect, the top rail from each ingot of that heat shall be rejected; and (*c*) a second test shall then be made of a test piece selected by the inspector. If this second test piece breaks, the remainder of the rails of the heat shall also be rejected. If this second piece does not break, the remainder of the rails of the heat will be accepted as either No. 1 or No. 2 classification, according as the deflection is less or more, respectively, than the prescribed limit. (*d*) If the test piece, test ('*a*'), does not break, but when tested to destruction shows pipe, the top rail from each ingot shall be rejected. The remainder of the rails of the heat will be accepted either as No. 1 or No. 2 classification, according as the deflection is less or more, respectively, than the prescribed limit. (*e*) If the test piece, test ('*a*'), does not break, and when tested to destruction does not show pipe, the rails of the heat will be accepted as either No. 1 or No. 2 classification, according as the deflection under test ('*a*') is less or more, respectively, than the prescribed limit. Note—The drop test and the deflection requirements may be modified if considered advisable by the railroad company.

"15. The drop-testing machine shall have a tup of 2,000 lb. weight, the striking face of which shall have a radius of not more than 5 in. The anvil block of the drop-testing machine shall weigh at least 20,000 lb., and the supports shall be part of, or firmly secured to, the anvil. The foundations for the anvil block shall be such as will meet the approval of the railroad company.

"16. No rails shall be accepted which contain any physical defects that impair their strength.

"17. Rails improperly drilled or straightened, or from which the burrs have not been properly removed, shall be rejected, but may be accepted after being properly finished.

"18. No. 2 rails to the extent of 5 per cent. of the whole order will be received. All rails accepted as No. 2 rails, must have the ends painted white, and must be kept separate from No. 1 rails, and be shipped in separate cars.

"19. Rails will be accepted and billed according to actual weights.

"20. All rails must be loaded in the presence of the inspector.

"21. The makers shall furnish the inspectors with the carbon determination of each heat, and, also, two complete analyses which shall represent the average steel of each day's work, before the rails are shipped. These analyses will be checked from time to time by the railroad company's chemist, and, on request of the inspector, the makers shall furnish a portion of the test ingot for check analyses.

"22. Inspectors representing the railroad company shall have free entry to the works of the makers at all times when the contract is being filled, and shall have reasonable facilities afforded them by the makers to satisfy them that the finished material is furnished in accordance with these specifications. All tests and inspection shall be made at the place of manufacture, prior to shipment."

To still further strengthen the future practice of the company and to insure that whatever changes are made shall be based upon accurate data, a revised and uniform system of blanks and reports of wide scope has been devised whereby a full history of all rails purchased by the company shall be fully recorded. One series of blanks will cover information obtained at the mill during the process of manufacture. A second series will relate to rail statistics to be prepared by division officers, including a complete record of rail failures in main track. A third series will provide for statistical compilations of rail failures. A final series is to provide reports of actual measurements of rail sections laid in the track at stated periods, in order to determine the relative wear of rails of different composition and manufacture. These reports, together with the very careful inquiries and observations of the standing committees appointed to keep continually in thorough touch with the art of rail making, are expected to supply the company with data that will enable it to secure the best rails which, at the time of any order, it is possible to manufacture.

The Pennsylvania committee considered very carefully the advisability of changing the weights of rail sections from those which have been the standard of the system. It had been suggested that the increasing severity of service would probably call for the use of a rail of greater weight per yard. The committee concluded, however, that the sections should continue to be of the same weights as those already in use, although a change of radical character was made in proportioning of metal to the different parts of the rail. The design in these changes has been to so apportion the metal as to reduce to a minimum the strains encountered in manufacture—which often have injurious results upon the rails—and to secure the greatest possible resilience and wearing qualities.

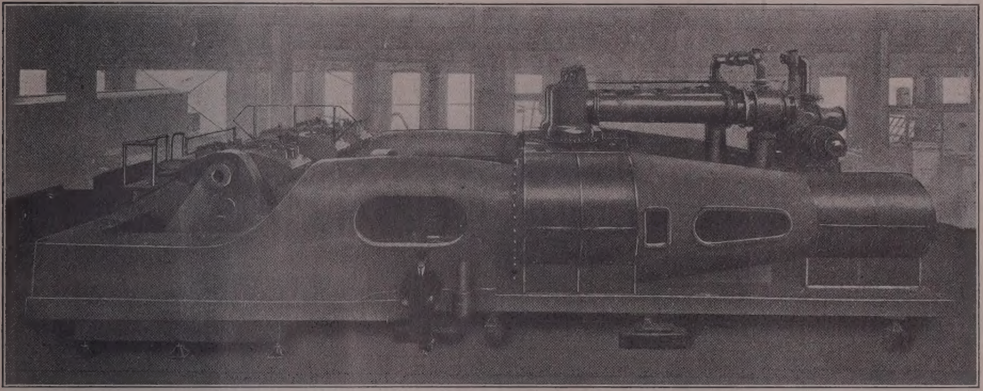
THE RECOVERY OF A LOCOMOTIVE of the Pennsylvania Railroad, which, in a derailment, had rolled some 200 ft. down the side of a mountain, was recently accomplished in an unusual manner. The engine had rolled so far from the track, and the side of the mountain was so steep that all attempts to raise it by two 50-ton wrecking cranes proved unsuccessful. The locomotive was finally righted and turned around by means of hydraulic jacks, and then moved by means of a temporary track along a wagon-road some 200 ft. to the foot of one of the historic inclined planes up the mountain side which Stephen Girard had graded some 80 years ago as part of his plans for a railroad between Danville and Pottsville, Pa. Although the incline is at an angle of about 45 deg., and is 350 ft. long, a temporary track was laid up it to the main line, and the locomotive hauled up it on its own wheels by means of the two 50-ton wrecking cranes. A heavy steel cable in five sections was run independently from each crane to the locomotive to permit either of the cranes to hold it on the incline in case of accident to the other.

A 25,000-Horse-Power Reversing Engine for a Blooming Mill.

The enormous reversing engine, rated at 25,000-h.-p., which the Allis-Chalmers Co. has been constructing for the South Sharon works of the Carnegie Steel Co., is probably the most massive, heavy engine ever built. It is a horizontal twin-tandem compound-condensing machine, coupled directly to a large blooming mill, and its progress through the builder's shops has attracted much attention not only on account of its size, but also on account of the shop methods which were adopted in carrying on the work. Its size and massive character are shown by the accompanying reproduction of a photograph of the assembled engine in the builder's shops, while the drawings show the general features of the design.

The high-pressure steam cylinders are 42 in. and the low-pressure cylinders 70 in. in diameter, and the stroke is 54 in. The high-pressure cylinder has one piston valve and the low-pressure cylinder two piston valves, and each end of each cylinder is fitted with a relief valve to free the cylinder from water.

The engine frames are probably the most massive castings ever made for an engine, one of them weighing 105 tons and the other 115 tons. The frame and slide are combined in one casting. The frame next to the roll train has two bearings, while the other frame has but one, as shown in the drawings. A plate is cast in the bottom of each frame to form a receptacle for oil thrown off by the crank pins, journals and slides. Each bearing is fitted with babbitt-lined, adjustable shells for taking up wear.



The 25,000-Horse-Power Engine Assembled in the Builder's Shops.

The frames and cylinders rest on heavy, deep-ribbed sole plates and the frames are tied together by a heavy plate rigidly bolted to their lower edges. A bead is cast on the outer edge of the sole plates so as to catch all oil and prevent it from running down into the foundations. All joints in the sole plate are tongued and grooved.

The crank shaft is made of open-hearth hammered steel, the double-throw crank being forged solid with the shaft. The shaft and the pin of the double-throw crank have hollow centers. The outer crank and its pin are also of open-hearth hammered steel, forged together and pressed onto the main shaft. The counterbalances are of cast steel and hook over projections provided on the ends of the double-throw cranks.

The main crossheads are of cast steel and fitted with babbitt-faced, removable and adjustable shoes, turned and scraped to fit the bored slides. The crosshead pins are fitted to the crossheads with a taper fit and can be readily removed without dismantling the engine. The connecting rods are of forged open-hearth steel and are provided with bronze boxes having wedge adjustment for taking up wear.

The piston rods are made of best open-hearth, hammered steel. The low-pressure rod is hollowed and keyed to the main crosshead. The piston spiders are of cast steel. The weight of the low-pressure piston is carried on the hollow piston rod.

The reversing gear is of the single eccentric, Marshall type. The connections transmitting the motion are carried between double levers so that the strain is always central. The reversing gear is operated by a steam cylinder in connection with an oil locking cylinder which follows the motion of the operator's hand lever. The high- and low-pressure cylinders are each fitted with throttle valve which operates simultaneously, and is handled by means of a steam cylinder with accompanying locking oil cylinder.

A Continuous Unloader.

The Western Pacific Ry. has two large fills near Altamont, Cal., which were made by a device called a continuous unloader, which does away with a construction trestle and at the same time allows the dumping to be done from the level of the finished sub-grade. The idea was proposed by Mr. F. F. Lloyd, chief engineer of construction, and the results are said to have been very satisfactory, while the apparatus is considerably cheaper than a trestle.

The unloader consists of a platform built of heavy 30-ft. timbers radiating from the base of a large mast, the outside ends of the timbers being supported by iron rods from the top of the mast. There are 24 of these radial timbers or spokes, each 8 x 17 in., set on edge, and beneath the spokes are large cross-beams which make the apparatus more rigid and render it possible to be moved forward. The cross-beams ride upon iron rollers, which in turn roll on sills placed on top of the fill. Around the outer end of the 30-ft. timbers is laid a circular track of 3-ft. gauge, built of 9-in. grooved street-car rails securely fastened to the radial spokes. The whole affair is built of standard material which is not difficult to secure, and which could be used for other purposes if it were decided not to use the unloader again. The fact that the unloader can be taken apart and moved from one piece of work to another is one of its advantages over a temporary construction trestle. In the middle of the track is a cable operated by a small donkey engine built on the platform of the unloader between the receiving and delivery sides of the circular track. From one drum on the engine the cable passes around the main sheave and around the track, between the rails, to a second drum.

When a train of cars is brought to the unloader from the cut, the engine is uncoupled, and the cars are fastened to the cable by means of a simple hitching device. They are pulled around the track, dumped, righted and then delivered to the waiting engine without having stopped. It has been found that a man can dump eight cars a minute, and the capacity of the unloader is limited only to a solid train of loaded cars, one right after another all day long. Since the operation of the unloader, all the cars that have been furnished by a 70-ton shovel have been dumped with ease, and the greatest run has been 544 cars in 7½ hours. The material may be dumped at any point around the unloader, and it is possible to carry the fill to a width of from 40 to 70 ft. The force required about the unloader consists of a carpenter and three laborers. They attend to the moving, jacking up and blocking. To unload the trains requires an engineman and four laborers. The unloader is moved forward on the rollers over the newly-made ground about 10 ft. at a time, the donkey engine doing the moving. It has been found that it can be moved forward the necessary distance during the noon hour without impeding the work. A description of the apparatus, which has been patented, was given in the "California Journal of Technology," from which these notes are taken.

Subsurface Investigations on the Catskill Aqueduct, Board of Water Supply.

A paper by Robert Ridgway, Department Engineer, Board of Water Supply, before The Municipal Engineers of the City of New York.

General.—The Northern Aqueduct Department, one of the five field departments into which the work of the Board of Water Supply will be divided, includes that portion of the Catskill Aqueduct between the Ashokan Reservoir and the Croton Watershed, and has a length of 60¼ miles. The work assigned to this department is the location and construction of the aqueduct to the Croton Watershed so that Catskill water may be delivered into Croton Lake at the earliest practicable date and before the completion of the works nearer the city. In his report to the Board of Water Supply on Oct. 7, 1905, the Chief Engineer, Mr. J. Waldo Smith, called attention to the fact that the combined capacity of the old and new Croton aqueducts is in excess of the yield of that watershed in a dry year. If, at such a time, the deficiency can be supplied from Catskill sources and the full flow maintained in the aqueducts the danger of a water famine in the parts of New York City supplied with Croton water will be averted.

Attention is called to the extraordinary size of this aqueduct as an engineering structure. Where cut and cover construction is used the conduit will have a horseshoe section 17 ft. high by 17 ft. 6 in. width. This section is approximately 3½ ft. higher and 4 ft. wider than the normal section of the new Croton aqueduct tunnel. The capacity of the latter is about 300 million gallons per day, while that of the Catskill aqueduct in the Northern Aqueduct Department is about 600 million gallons per day. In tunnels the section is somewhat smaller, due to the greater slope allowed.

The aqueduct location has now been fixed, but before this definite line was laid down on the ground a careful study was made of all the general routes which showed any promise of being feasible. The extreme lines crossed the Hudson River over twenty miles apart. The locations were first studied on the topographical maps of the United States Geological Survey and those which seemed to require it were further studied in the field. In many cases stadia or plane table surveys were made and the topography plotted on scales from 400 to 100 ft. to the inch. Comparisons were then made of the different routes, the most important factors considered being economy and speed of construction, and safety in operation and maintenance. After careful consideration, one after another of the routes was eliminated and the present one adopted as the best, all things considered. In connection with these preliminary surveys it is interesting to note that 57,000 acres were covered by the topographical surveys.

The United States Geological Survey maps mentioned were of great assistance in the preliminary work, and it is impossible to estimate how much time and money were saved to the city because they were available. The broad area covered by them made it possible to review all possible routes without extensive investigations of every one of them in the field, so that studies were concentrated on those most feasible. If they, or similar maps, had not existed there would always have been a doubt whether the route selected was the best one. It is a pleasure to testify to their general accuracy.

The ideal location would be one where the aqueduct would be constructed in open cut for its entire length on the hydraulic gradient, and on a direct line. Open cut, or as it is called cut and cover, construction was therefore sought. Such an ideal location, however, it was manifestly impossible to find and efforts were devoted to ascertaining that one which would give the smallest percentage of tunnel at hydraulic grade and siphon below it without undue increase in the length of the line. It is obvious that where the conduit is constructed below the grade it must be strengthened to withstand the increased pressure, thus adding largely to its cost and to the difficulties of construction, operation and maintenance.

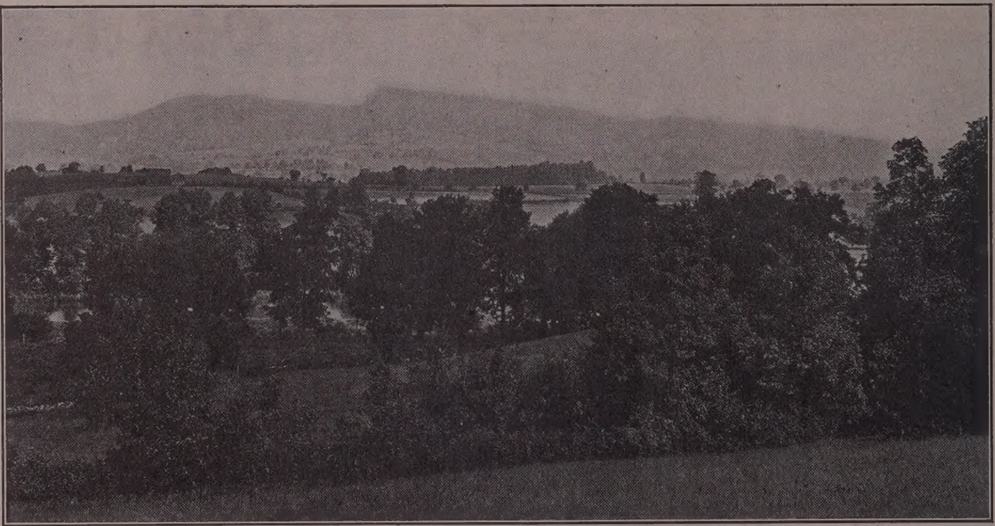
For the aqueduct at grade the cut and cover type of construction is preferred because it can be more economically and rapidly constructed than a tunnel, which is only used in going through a hill, instead of around it, when it cuts off enough length to effect a saving in cost. The aqueduct as now located is divided as follows:

Aqueduct at grade in cut and cover	36.35 miles,	60%
Aqueduct at grade in tunnel	6.66 miles,	11%
Aqueduct below grade in siphon	17.25 miles,	29%
Total	60.26 miles,	100%

The deepest depression encountered is the gorge of the Hudson River, the main drainage channel of the region, which, of course, must be crossed with any location that could be adopted. West of the river the general direction of the line is necessarily across the drainage of the country and in addition to three of the minor streams three wide valleys, the Rondout, the Wallkill and the Moodna, will require siphons each from 3½ to 4½ miles long. East of the river also several tributaries of the Hudson must be crossed by siphons from 600 ft. to 1½ miles in length. The following are all of the siphons of the department, arranged in geographical order from north to south, with the probable type of construction and the approximate length of each: Esopus, steel pipe, 1,850 ft.; Tongore, steel pipe, 700 ft.; Rondout, rock tunnel, 23,610 ft.; Wallkill, rock tunnel, 23,400 ft.; Washington Square, steel pipe, 3,550 ft.; Moodna, rock tunnel, 19,800 ft.; Hudson River, rock tunnel, 4,450 ft.; Foundry Brook, steel pipe, 3,800 ft.; Indian Brook, steel pipe, 600 ft.; Sprout Brook, steel pipe, 2,270 ft.; Peekskill Creek, steel pipe, 7,040 feet. The total length of the siphons is 91,070 ft.

It is possible to cross the two smallest of these valleys by means of a viaduct at grade, but the higher cost of this construction and the trouble it might be expected to give in maintenance led to the rejection of this type. Several other types have been considered, notably, deep tunnel in rock, steel pipes, reinforced concrete pipe. Of these the deep tunnel is preferred where the rock underlying the valley is not too deep or of too uncertain a character, because it is cheaper to construct and give greater assurance of safety and durability.

Experiments have been made with reinforced concrete pipes with a view to adopting this form of construction for some of the shallow depressions in place of steel pipes if the results prove favorable. A number of 5-ft. pipes were made and tested in the yard at the foot of West 79th St., and at Hunter's Brook, at the south end of the department, an experimental pipe 11 ft. in diameter and 210 ft. long was constructed last season and is to be tested under varying pressures during the coming spring. The results of these tests will determine to what extent this type of structure will be used, and if successful the pipe already built at Hunter's Brook will form part of the aqueduct.



View Across the Wallkill Valley on the Line of the Catskill Aqueduct.

At each siphon, except a few of the smaller ones, extensive investigations by means of borings were necessary before the definite location of the aqueduct or the type of structure could be determined. The services of geologists were enlisted from time to time to examine and report on the conditions in the light of data obtained from the investigations, and their expert advice has been of great assistance in reaching a decision in each case. In all of the larger valleys there exist preglacial gorges worn in the bed rock by ancient streams, generally masked by such a heavy cover of drift that their location, depth and extent could only be ascertained by the most careful search with core drills. It was early found that wash borings were utterly unreliable for this work as they could not penetrate the boulder ground and the only information they gave was of a negative character.

It is essential that a siphon tunnel must be deep in sound rock on account of the great hydrostatic pressure to be considered. It will be seen then that the elevation at which such a tunnel should be constructed is fixed generally by the depths of the gorges. The latter in all of the larger valleys have now been definitely determined with the single exception of the Hudson River where the difficulties of carrying on the explorations have been exceptionally great as will be described later.

While the explorations of the preglacial gorges and the determination of the rock profile for the entire length of each siphon has been of primary importance it has likewise been necessary to investigate carefully the general geologic conditions. Twelve formations of rock are met with in the department, all but two of which must be penetrated by the siphon tunnels. Nearly all of them are good rocks to tunnel and their characteristics are well known to tunnel contractors or miners. There are exceptions, however, to be noted later.

To date the total length of wash borings made for the work is 51,255 ft. and of core borings 55,774 ft., total 107,029 ft. or 20.27 miles. This work was done by ten contractors and by a small force employed directly by the Board of Water

Supply. The borings range in depth from a few feet to a maximum of 656 ft., the latter hole being in the Rondout valley. The deepest hole made in the Hudson River to date was 617 ft. deep measured from the river surface.

Boring Methods and Machines.—The material drilled in the explorations of the Northern Aqueduct Department may be divided into two general classes, surface and rock. The surface, or glacial material, consists of varying mixtures of sand, clay, gravel and boulders. This material often covers the rock to a depth of several hundred feet. The rock over the whole department may be regarded as distinctly hard, that is, it is not of a quality to be excavated without blasting.

In many respects boring through earth, or surface, is more difficult than boring through rock. If the rock is of a uniform quality, a progress of 10 to 30 ft. a day can often be maintained, but the presence of a gravel bed or boulder in the surface material may bring on troubles which cause a delay of weeks in progressing a few feet.

The difficulties of boring in rock are mostly due to the occurrence of seams, caves, or material so soft that the walls of the hole cave in. In a few cases where rocks are extremely hard, the diamonds and other cutting agents wear away rapidly, but in general the troubles in rock are due to the presence of soft spots.

While the deep surface holes are usually the hardest to drill, under some conditions they are very easy. In sand or clay free from boulders the holes often go down with astonishing speed. In certain cases, 150 ft. a day has been made and a hole over 200 ft. deep to rock finished in two or three days. Again a hole of the same depth in heavy gravel or boulder ground usually takes six or seven weeks. The deeper such a hole goes, the harder it becomes to drill it. The experience in the Northern Aqueduct Department indicates that in the material encountered more rapid and consistent progress can be made in drilling rock than in drilling surface material.

There are four general types of drilling machines: 1. Wash drill, 2. Spudding machine, or Churn drill, 3. Diamond drill, 4. Calyx, or Shot drill. The two first are used in drilling surface material, and the three last for drilling in rock. There are a great number of makes of each type of machine, and many drilling rigs use a combination of drilling methods, depending upon the ground penetrated.

Wash Drills.—The object of a wash boring is to get a casing pipe down through the soft surface material to rock. The rig consists of a derrick and winding drum, operated by hand or steam engine. The hand rigs have limited penetration, except in very easy ground, and a small boulder or cobblestone will stop the boring. The heavy steam machines for drilling through several hundred feet of glacial drift in exploring for rock, are used in connection with diamond or shot drills.

The casing is put down by washing up the sand and clay at the bottom end, and driving it into the cavity thus formed. The ground is loosened by means of a line of drill rods or 1-in. pipe working inside the casing. This wash pipe carries a chopping bit at the lower end, and the line of rods is raised and lowered with a churning motion by means of the hoister. The cuttings are washed up inside of the casing by a stream of water constantly forced down by a steam pump through the inside of the wash pipe.

The casing is used in lengths of 6 ft. to 10 ft., connected by couplings, and is driven by means of a heavy weight operated by the hoister, after the manner of a pile. As a general rule, it is not possible to put the casing down easily much over 100 ft. on account of the skin friction of the outside material. When the limit of easy driving is reached, a smaller size is telescoped inside the first casing until it is in turn stopped by the friction. In some cases 6-in. casing is started, then 4-in., 2½-in., and occasionally 1⅝-in. casing is used in going down about 250 ft. Ordinarily in deep holes up to 350 ft. three sizes of casing are used. In driving casing a heavy blow should not be struck by dropping the weight a great distance, but the casing is tapped down by light, repeated blows, which loosen the outside sand and break the skin friction. At the same time, the casing is turned by hand with chain tongs, and the wash pipe is kept working.

In sinking casing in sandy ground, the progress is usually very fast, as the material loosened by the chopping bit can be rapidly washed up by the stream of water forced down the wash pipe. In such ground the casing sinks of its own weight, and does not have to be driven all the time. In very fine loose sand the wash pipe is often dispensed with; the pump is connected directly with the top of the casing so that the water is forced to the bottom of the latter and comes up on the outside while the casing is constantly turned with chain tongs. This operation is called "washing down" the casing. Where gravel is encountered, progress is much slower, as the stones have to be first chopped up and then the cuttings washed out. In porous ground it is necessary to keep the casing driven down very close to the end of the wash pipe, or the water will all be lost in the ground instead of returning to the surface carrying the cuttings. In gravelly ground carrying cobbles, it is usually best to make sparing use of the wash pipe, as the fine sand is washed away and a nest of cobble stones collects under the foot of the pipe.

Steam Wash-Boring Outfits.

When the stones or boulders are too large to be readily chopped up, dynamite is used to break them up or shoot them to one side. The slowest progress is made in such ground, as it is necessary to pull up the casing a few feet every time a shot is fired. When a large boulder is encountered, a hole is drilled through it by the diamond bit, and dynamite used to break it up.

The best way to pull casing is not by a steady strain, but by tapping it upward with a heavy weight, as in driving. When the casing is struck, it is pulled by putting a strain on it by means of two hydraulic jacks, and then tapping upward on it with the weight. It seems to require much more power to tap upward than downward. A weak machine can slowly lift a heavy weight a certain distance and let it drop by gravity, while the upward tap depends upon the direct upward pull of the machine aided somewhat by the momentum of the weight.

Troubles in Drilling Surface Material.—The principal troubles in drilling surface material arise when gravel or boulders are met. A boulder in sand under the bottom of the casing is a hard thing to remove. The sand washes up inside the casing, making it hard to chop the boulder. Even if the boulder is shot to one side, it is liable to follow the casing down, and drop into the hole washed out under the pipe. When dynamite is used it is lowered into the hole on top of the boulder. The casing is then raised to keep it from being broken by the explosion. The sand sometimes flows in as the casing is raised and floats the dynamite up inside the pipe, which is broken by the shot. When this happens, a new hole has to be started. Dynamite in the ordinary stick form is used for blasting boulders. It is let down into the hole by means of the battery wire and exploded with a magnet battery. Experience seems to indicate that the grade of explosive known as "gelatine" is more reliable in its action than dynamite, and that the liberal use of exploders, one to each stick, and of a powerful battery, is essential.

One trouble in dealing with boulders is the difficulty of exploding the dynamite. There seems to be some strange unexplained difficulty in setting off dynamite in the bottom of deep holes, which is usually attributed to the chilling of the explosive. After a miss, the dynamite has often to be chopped out. This can be safely done in a hole as shallow as 30 ft. in depth. The failure of the dynamite to explode is sometimes due to a leakage of current occurring between the battery wires, which are surrounded by water in the hole, but sometimes the end is blown off a stick without exploding the whole of it. Even when the casing is pulled up high enough to clear the dynamite by several feet, the shock of the explosion communicated through the water has been known to split the casing far above the boulder.

Gravel of varying size is one of the most difficult materials to penetrate. It often leads away the wash water, and also puts great strain upon the wash rods, due to wedging the bit and causing the rods to part. In such a case a conical recovering tap is let down on a line of rods and cuts a thread into the end of the broken pipe, so it can be raised. If a hole in the rods cannot be found by the inside tap, an outside recovering tap is let down, and a thread cut on the outside of the rods, so they can be picked up. Gravel frequently causes the loss of a chopping bit, which is broken off by the great strain and twisting effect to which it is subjected. A lost chopping bit may cause no end of annoyance, as it blocks the bottom of the pipe, and cannot be easily picked up with a fishing tool. The lost bit is treated like a boulder and blasted repeatedly to drive it to one side. A chopping bit has been known to follow the casing down 40 or 50 ft. below the point where it was lost, giving trouble all the way. The ground is often so soft and watery that fine sand flows up inside the casing, sometimes causing the wash pipe to become wedged fast, so that the casing has to be pulled up before the wash pipe can be removed. When the material is at all difficult, the wash boring machine should be equipped with a powerful pump and hoister. The lack of adequate equipment is the principal source of trouble with such rigs. A strong pump will make it possible to penetrate and wash out gravel where a weak one would be useless. A strong hoister is essential in driving out casing and in chopping gravel.

One great source of trouble is the parting and bending of casing, both in pulling and driving. This is due to the tendency to save money in the cost of material by using too light a pipe. The weight of casing known as "extra heavy" should always be used in bad ground. Moreover, the casing must not be driven too hard. It must be coaxed down, rather than forced. It is rather by a succession of light, rapid blows that casing and wash pipes are driven than by the use of great force.

Churn Drill.—These machines are used for drilling in either glacial drift or rock. They are used extensively for drilling wells for water, and were employed by the Board for exploring the depth of glacial deposits above the bedrock. They do not give a rock core like the shot and diamond drills, but the material comes to the surface in fine fragments.

The machine consists of an apparatus for raising and lowering, with a churning motion, a string of tools attached to a rope. Only a small quantity of water is required to make a thick creamy batter of the rock as it is broken. Drilling is stopped and the hole bailed out from time to time by a sand pump, consisting of an iron pipe with a valve on the bottom, which is lowered into the hole.

The string of tools consists of a hardened steel chopping bit, screwed into a stem from 10 to 40 ft. long. A set of jars is attached at the top of the stem, for jerking or jarring the bit loose when it sticks in the bottom of the hole. The entire string of tools used on holes about 400 ft. deep, weighs about 1,000 lb.

This machine is cheap in operation, as only two men are needed when drilling in rock. When driving casing, four men are needed. As very little water is required, the churn drill has a great advantage where water is scarce, but is often at a great disadvantage in a wet hole, due to the fact that the cuttings are removed only at intervals.

Diamond Drill.—This machine is used to take a sample or core of the rock penetrated. The machine consists of a geared spindle driven by hand or steam power which rotates a line of rods and feeds them down the hole. The cutting is done by the diamond bit screwed on the bottom of the line of rods. This bit has an annular form, and the diamonds are set in the soft metal around the edge in such a manner as to cut a small clearance on the inside and outside. Eight stones are ordinarily used. Care is taken that the diamonds cut the whole area of rock at the bottom of the bit, otherwise the steel would be worn and the stones loosened.

As the bit cuts the rock the core passes up through the center of the bit into the core barrel, an especially straight piece of pipe a little larger in diameter than the rods, where it is held by a spring. Water is pumped down the rods to keep the diamonds cool, and to wash away the fine cuttings.

Black diamonds from Brazil are used for the bits. Before being put on the market the stones are crushed, and the large pieces with sharp edges selected. The size varies from one to two carats, worth from $40 to $90 a carat, according to size and quality. The stones vary greatly in grade, some wearing away rapidly, while others prove almost indestructible. A good set of stones well tried by use is often worth from $500 to $800.

Troubles with the Diamond Drill.—If a seam in the rock carries away the water, the cuttings are liable to pack around the core barrel, or the bit may become wedged in an open crevice in the rock. In either case, the rods may be twisted off, and have to be fished for by the recovering tap. There are a number of other devices for recovering rods, but at times they all fail. A bit was lost on the Esopus division in a 20-ft. cave. The cave contained a stiff clay which at first gave support to the rods, but as it was washed away, the rods bowed out and broke off. The top of the broken rod was so much out of line that it could not be picked up, and was never recovered. The contractor asserted that the value of the lost bit was $1,000.

A great deal of heat is generated as the bit cuts. If for any reason the water does not get to the bit, and the boring continues, the diamonds will be burned and completely ruined. Therefore a drill runner watches the behavior of his pump and the flow of water from the top of the casing.

If a bit is used too long, the steel is worn away from the diamonds so that the stones are torn out and ground up by the bit.

At times the cores in the core barrel break up, and pieces become wedged, causing the bit to bind. When this blocking occurs in very hard rock diamonds are often torn out and shattered.

Shot Drill.—The shot or calyx drill uses rods and core barrels similar to the diamond drill except that the core barrel is generally larger in proportion to the size of the rods. It is fitted with a bit at the lower end with a single large notch cut in it. Hardened steel shot is fed to this bit along with the water and cutting is accomplished by the grinding of the shot below the bit. The core barrel is provided at its upper end with a calyx extension open at the top, equal in diameter to the core barrel. The calyx surrounds the rods, and acts as a receptacle for catching the fine cuttings. If sufficient water is used to bring the cuttings to the surface, it washes the shot away from under the bit. Therefore a small amount of water is used and the cuttings are allowed to wash up and collect within the calyx. They are in this way brought to the surface, When it is desired to bring the core to the surface, it is grouted with coarse sand or gravel. A few handfuls of this material are poured into the rods and washed down. This sand or gravel becomes wedged between the core and core barrel and holds the core in the barrel.

Shot drills are good where there are uniform rocks to be bored, such as Manhattan schist. They have been used on a large scale in New York to drill holes for plunger elevators. They seem to be less efficient in drilling stratified rocks of varying hardness. The progress on the Esopus division of the work with them has been 2 or 3 ft. per day as compared with 10 to 20 ft. with diamond drills. There is one bed of rock there, the Shawangunk grit, 250 ft. or more thick, which the shot machine seems to be absolutely unable to penetrate. This rock is pure quartz conglomerate and contains seams in many cases having water flowing through them. When the shot bit reaches one of these seams in the grit the shot is carried away and the bit is left to grind on the quartz. No steel however hard will make much impression on it by rotary action although it can be readily drilled by percussion drill.

Interpretation of Boring Records.—To the engineer unused to such work it is difficult to judge whether a boring contract is being handled properly. The tendency is to magnify the difficulties encountered when the trouble may lie with the contractor's methods or appliances. Months may be spent in gaining only a few feet at great labor, and anybody watching the operation might conclude that the difficulty was insurmountable. The substitution of a different machine or of men more efficient in this class of work means in nearly every case the successful completion of the boring. The experience of the drill runner is, of course, a very large factor in successful boring, but his experience alone cannot make up for inferior machines or equipment. Such troubles as the bending or breaking of the casing is frequently attributed to the difficult character of the ground, but the employment of heavy casing or proper machines will usually overcome the difficulty. The same considerations apply to borings in rock as in earth. A shot machine may be employed to do work for which it is entirely unsuited, or a small diamond drill, not of the hydraulic type, may be found inadequate for a certain rock. In one case it was found that a light machine could make only 2 ft. per day in the grit whereas a hydraulic rig easily made about 10 ft.

As stated above, misleading conclusions are apt to be made from boring work and records. Where a boring machine is having great trouble due to seams in the rock, one is apt to conclude that the rock itself is very hard, whereas such a conclusion may not follow at all. For instance, a rock in which the strata are in a vertical position may be very difficult to bore with a diamond drill; such a rock is in just the right position for tunneling through, as the layers will support themselves instead of caving in flat slabs, which might happen were the strata in a horizontal position. Seams in rock will bother a rotary drill greatly, whereas a percussion machine may readily penetrate it. Moreover a rock may be penetrated very easily by a rotary drill and not by a percussion machine.

Reliance cannot always be placed upon the percentage of core recovered from different rocks. These percentages are apt to vary according to the variety of drill and manner in which the machine is handled. The dip of the strata is an important factor; slate on edge is very easily tunneled into, but may yield a very low percentage of core, whereas material difficult to tunnel may give a high percentage of core.

It is most difficult properly to interpret wash boring samples. If clay or uniform sand is being penetrated the samples are all right, but in the majority of materials only the coarser particles are apt to be preserved unless great care is used. Where hardpan, composed of a large percentage of clay, is being penetrated, in drilling it will cause the loss of most of the clay in the wash water, so that the little material recovered is composed of sand and gravel. Such a sample may be labeled "Sand and Gravel" by the drillers, and an examination of the contents of the sample bottle will apparently bear out this erroneous conclusion.

(To be continued.)

A 50-Ton Steel Derrick Car has recently been built for the Bridge and Building Department of the Chicago, Milwaukee & St. Paul Ry. The car is 51 ft. long and the trucks are spaced 40 ft. on centers. The boom is of structural steel and adjustable in length. By the use of various combinations of 15 and 21 ft. sections, the boom can be made 42, 57, 65 or 80 ft. long. The capacity of the boom when at its maximum length of 80 ft. is 10 tons, and its swing with the outriggers in use is 26¾ ft. from the center line of the track or 12 ft. without the use of the outriggers. With the shortest length of boom, 42 ft., the capacity is 50 tons, which can be swung 15½ ft. from the center line with the use of the outrigger and 6¾ ft. without its use. The lower end of the boom swivels in a ball-and-socket joint. The boom tackle is composed of two 7-sheave blocks with ½-in. sinde plates and 15 parts of ⅞-in. steel wire rope. The sheaves are self lubricating, 24 in. in diameter and have phosphor-bronze bushings. The tackle supporting the 50-ton hook is composed of a 5-sheave and a 4-sheave block, having 7/16-in. side plates and 10 parts of ¾-in. wire rope. The sheaves are 21 in. in diameter. The A-frame when erected for operation stands 21 ft. above the top of rail. It can be folded back by revolving on the casting at its base so as to reduce the height from top of rail to the top of the frame to 16 ft. for shipment. The outrigger, when in place, has its bearings 5 ft. 9 in. from the center line of the track. The cab contains a 50-h.-p. hoisting engine with two drums and four spools. A propelling device is provided which can propel the car forward, carrying an 80-ft. girder at the end of a 57-ft. boom.

The Canal Street Tunnel Sewer, New York City.

The construction of the new subway loop that is now being built in New York City to connect the Manhattan terminals of the Brooklyn and Williamsburg bridges with the present subway lines, has necessitated important rearrangements of the sewer system in the district traversed by the new line. When the present subway lines were built the sewerage of this same district was arranged so that the storm water and sewage flow of a considerable area north of Canal St. and east of the subway was carried in a 5-ft. 6-in. trunk sewer south in Centre St., then across Leonard St. and Mulberry Park to Park St., and thence through a 6-ft. 6-in. outfall sewer to the East River.

The new subway will occupy almost the entire width of Centre St. and the portion from Worth St. to Broome St. is to be a four-track structure with a spur deflecting into Walker St. and thence to the Manhattan bridge. Moreover, it is intended to build in the near future a two-track subway in Canal St. For these reasons, viz., lack of room adjacent to the subway wall in Centre St. and the necessity of a series of deep inverted siphons if the sewer were rebuilt on its present alignment, the latter course was not considered feasible.

Accordingly, it was determined to build a new sewer whose course should be south in Baxter St. from Grand St. to Canal St., two blocks east in Canal St. to Mott St., south in Mott St. to Park St., and west in Park St. to a junction with the outlet sewer that is at present carrying the flow to the East River. From Centre St. east to Mott St. the grade of the subway rises sharply while the grade of the sewer falls at the rate of 1 ft. in 1,000 ft. At Mott St. the sewer is so much below the subway that the former can be carried under the latter without a siphon. From Baxter St. east to Mulberry St., about 240 ft., the section of the sewer is circular, 7 ft. in diameter, and from Mulberry St. to Mott St., about the same distance, the section has vertical side walls and arched roof, the maximum dimensions being 7 ft. 6 in. for the height and 6 ft. ¾ in. for the width. This section is designed to carry very heavy loads, which will be referred to later. The crossing under the subway tracks is made with two 60-in. cast-iron pipes embedded in concrete. From the south side of the crossing to the junction with the outlet sewer, about 1,070 ft., a 7-ft. circular section substantially like that in Canal St. is used.

The new subway will be at present a two-track line but provision has been made for the ultimate installation of two additional tracks. In Canal St. the two tracks to be installed first will occupy the south side of the street. Between Baxter and Mulberry Sts. there is room between the curb lines for the four-track subway and the sewer, but east of Mulberry St. the street is much narrower and the only space available for the sewer is close alongside the north property line. The north side of Canal St. in the two blocks traversed by the sewer is closely built up with four, five and six-story brick and stone business buildings and the sewer work, which has preceded the subway construction has to be carried on almost directly under the front wall foundations of these buildings. This is being successfully accomplished without damage to the buildings by building the Canal St. section of the sewer entirely in tunnel, the tunneling being carried on by a system especially adapted to prevent settlement. The method used is a modification of what is known as the sectional shield system, sometimes called the Meem system. The principal features were worked out by Mr. J. C. Meem, and the system was adopted and first used by Mr. Jas. Pilkington, sub-contractor, in building the Chatham Square tunnel sewer for the old Rapid Transit Commission in New York (The Engineering Record, Jan. 26, 1901). Later it was improved and amplified and used under the direction of its designer by the Borough Construction Co. in building the 64th St. tunnel sewer in Brooklyn (The Engineering Record, Nov. 21 and 28 and Dec. 5, 1903), and other important work.

Section of Sewer Built in Tunnel.

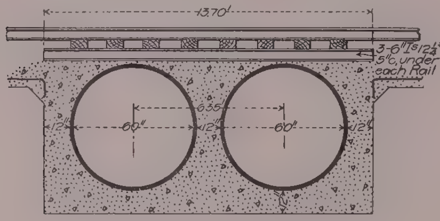
Underpinning of Building Adjacent to Sewer.

Circular Section of Sewer, Showing Timbering.

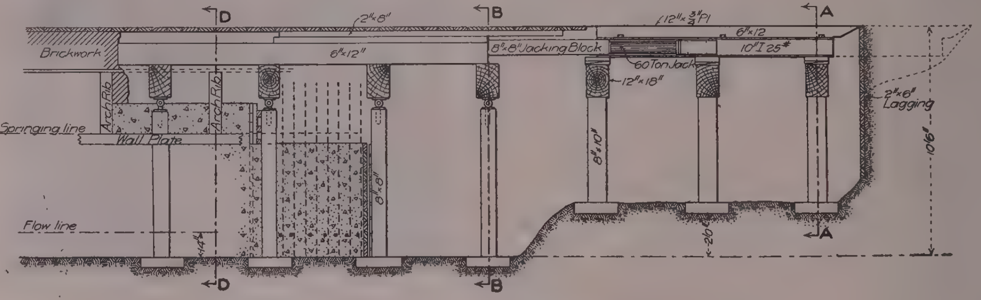
Two shafts have been sunk, one at each end of the Canal St. section. The Mott St. shaft is about 35 ft. deep and the Baxter St. shaft has a depth of about 20 ft. The grade of the sewer between the shafts is 0.1 per cent. and the street surface rises on a practically uniform grade from the shallower to the deeper shaft. The western part of this section could have been built in open cut, but Canal St. carries heavy vehicular traffic and it was deemed advisable to construct this as well as the eastern part in tunnel. Work is being carried on in two headings, one being driven from each shaft. The methods of construction employed for the circular section are substantially the same as for the arched section, and as the latter involves the carrying of much the heavier loads, it only will be described and illustrated.

This system of tunneling involves supporting the tunnel roof in the advance heading by wide sections of a roof shield, sometimes called "jills," forming an adjustable sectional hood. Each "jill" is driven ahead by a horizontal hydraulic jack. The complete set forms a continuous protection reaching from the permanent roof lagging to the face of the excavation. In this work three jills, arranged to conform roughly to the arch curve, are set side by side on bents of blocking about 5 ft. apart on centers. The load on each jill is carried mainly by two 10-in., 25-lb. I-beams, 10 ft. long, 18 in. apart on centers; their top flanges are fastened to three flat transverse bars to which in turn are bolted with countersunk bolts, three 6 x 12-in. yellow pine timbers about 12½ ft. long, beveled at the front end and faced with a ¾-in. steel plate wide enough to cover the whole width of the board and long enough to form a shoe over the beveled front end and a 3-ft. tailpiece in the rear. The I-beams rest on steel plates supported by blocking and wedges over each cap of the supporting bents. The wedges are used in connection with rocking blocks to vary the elevation of the shield section. One 60-ton jack, made by the Richard Dudgeon Co., placed between the I-beams, is used to operate each jill, acting in one direction against a diaphragm riveted to the webs of the I-beams and in the other direction against blocking and timbers that transfer the reaction to the finished masonry, the jack being connected with a high-pressure hydraulic test pump by a copper pipe, and operated by one or two men.

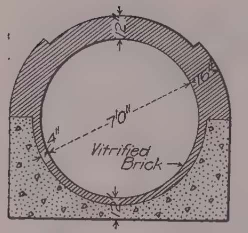
In commencing a heading three bents of blocking are set up, each bent consisting of two 8 x 10-in. battered legs, a 6 x 4-in. cap, and a "coffin" block, as shown in the drawings. The poling boards are then put in place and jacked into the face of the heading about 5 ft. ahead of the first supporting bent. At the same time the earth immediately under the shoes is dragged back into the heading. When the jills have all been advanced the excavation is continued further from the top downward, 2 x 6-in. lagging being very carefully fitted against the breast and along the sides as fast as possible to prevent all unnecessary disturbance of material. The soil in both headings is sand of varying fineness; hay is used to caulk the cracks. When the breast has been made tight another supporting bent is set up close to the lagging and side lagging is braced against the inclined legs in the manner shown in the accompanying drawings; the jills are then jacked ahead again. When the tailpiece begins to carry its earth load, transverse lagging planks are blocked up under it before each shove to sustain the earth after the tailpiece has moved ahead. This lagging has to be very carefully fitted to make tight joints. The outer ends of the outside lines of lagging rest on the side bars, but their inner ends and both ends of the middle set rest on two built-up longitudinal beams, each made of two 1 x 3 x 3/16-in. channels bolted together back to back with wooden separators between, forming a sort of continuous crown bar of very small dimensions.



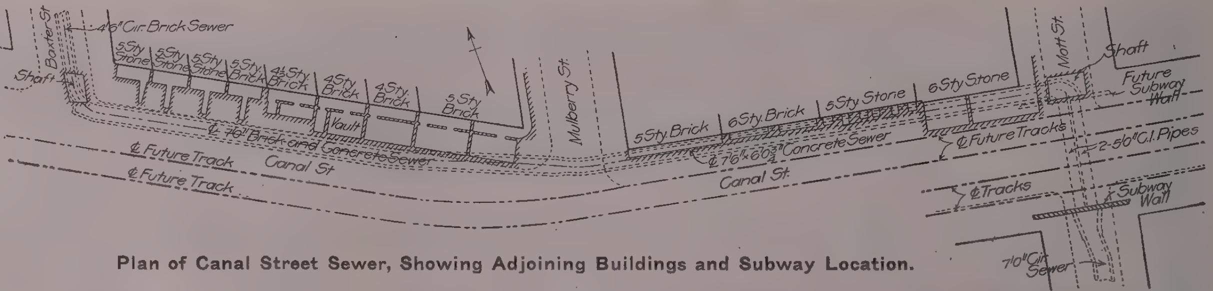
Cast Iron Pipe under Subway.



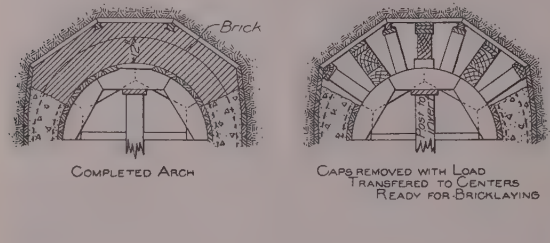
Longitudinal Section Showing Method of Driving Tunnel.



Circular Sewer Section.

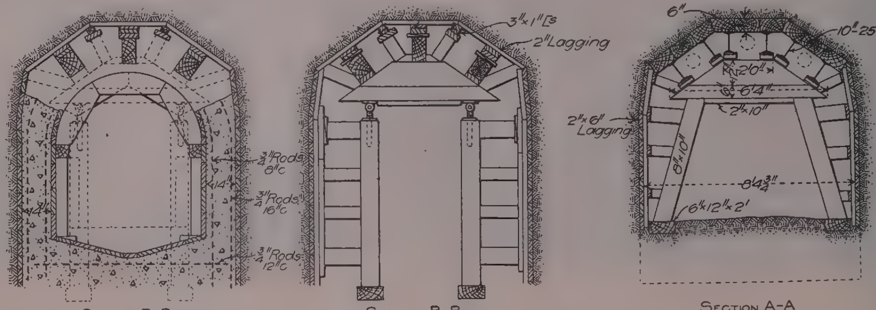


Plan of Canal Street Sewer, Showing Adjoining Buildings and Subway Location.



Completed Arch

Caps removed with Load Transfered to Centers Ready for Bricklaying



Section D-D

Section B-B

Section A-A

Progress in Timbering and Lining the Canal Street Tunnel Sewer.

In the advance heading the excavation is carried down to a level about 2½ ft. above subgrade. When this heading has proceeded a convenient distance pits are excavated to a level a little below subgrade between the legs of the temporary bents farthest from the working face and in these pits are placed bed blocks on which are seated 8 x 8-in. vertical posts to which the load on each cap is transferred permitting the removal of the temporary legs. Each vertical post is fitted with a jackscrew as shown in the drawings; these facilitate the adjustment of the posts and are much superior to the wedges commonly used. As fast as the interchange of posts goes forward the excavation is carried on to subgrade, additional lagging being added at the sides as required and braced against the vertical posts as shown in the drawings.

When a suitable length of subgrade has been exposed, wooden boxes are built loosely around the bottoms of the vertical posts and sand is packed in the space between post and box. Then the concrete for the invert is laid up to the level of the bottom of the walls, the reinforcement for the latter having been first fixed in position so that the lower ends of the vertical bars will be bonded in the invert concrete. The invert concrete is placed around the boxes before mentioned; these make it easy to remove the posts when this becomes necessary and the boxes are then broken out and the holes in the invert made tight with concrete. The surface of the newly laid invert is protected with plank as shown. A wide plank is then nailed vertically to one side of each post so that the upper ends of the planks are at the right elevation to support a wall plate with its upper surface at the springing line. The outside edges of the same planks engage the ends of the lagging that forms the inside surface of the sewer wall.

The concreting of the side walls is carried up to a little below the springing line at one time and this concrete is allowed to set before the second lift of concrete is added to bring the walls up to the elevation where the brickwork begins. Before the second lift of concrete is placed the wall plates are adjusted in position and between the timbering bents are placed wooden arch ribs to carry the arch lagging.

When the concrete is all in place and has set sufficiently, both sides of the arch are built up simultaneously with hard, common brick and 1 : 2 mortar of Giant Portland cement and sand. Theoretically, the thickness of the brick arch is 12 in., but to provide for possible settlement or variation from other causes the jills are kept above their theoretical position. All the space between the lower and upper lagging is filled with concrete or brick and mortar, as the case may be, and the arch accordingly varies from 12 in. to 16 in. in thickness. The blocking under the roof lagging is removed as the work progresses, but it is impracticable to remove the channel beams under the lagging and they are built into the brickwork.

The general practice in both headings has been to excavate both day and night, two 8-hour shifts being maintained, but muck is taken out only during the daytime; in the night it is simply pushed back from the working face out of the way. This leaves the tunnel unobstructed during the night, when all concreting and bricklaying is done. The excavated material is hauled to each shaft in a special dump car operated by hand on a 24-in. industrial track. These cars are 2 ft. wide, 4 ft. long and 2½ ft. deep. The front end is hinged at the top and the car body is pivoted so that it can be tilted forward, discharging the contents into a hoisting bucket lowered into a pit in the shaft below the level of the track. At the Baxter St. shaft a stiff-leg derrick operated with a hoisting engine is used to hoist the material. The Mott St. shaft is commanded by a cableway used in the open trench work on Mott St., which will be referred to later. In general, the excavating and timbering gang in each heading averages about eleven men. About 30 ft. advance per week in each heading is considered good progress, the advance in each heading being from 3½ to 7 ft. per day when working continuously.

The concrete is a 1 : 2½ : 4½ mixture of Giant Portland cement and sand and gravel secured on the work. All concrete is hand mixed, the mixing being done in the tunnel or above ground as is most convenient. Brick is stored in the completed sewer in high, narrow piles along both sides of the track; this is not only conveniently near the work, but it has the added advantage that the bricks are protected from the winter weather. The practice is to alternate the concreting and bricklaying gangs between the two headings, laying brick one night and concrete the next in each heading. In all the shifts above and below ground about 70 men are employed.

Notwithstanding the heavy loads carried by the tunnel timbering and the completed sewer no settlement of any of the buildings has occurred, the maximum settlement noted in the street being about 2 in. and confined to a space directly above the tunnel. The opinion has been expressed by engineers closely in touch with the work that the tunneling system described could be depended on under favorable conditions for executing all the work without additional safeguards. However, as a precautionary measure, some of the heaviest buildings have been underpinned with concrete piers extending down from the footings of the old brick and stone piers to a level 1 ft. below subgrade of the sewer. These piers are in general about 4 ft. square and are slightly enlarged at their bases. Steel needle beams through the old piers carried the load while the new piers were being built and grouted up to a solid bearing under the old pier footings in the usual manner. The underpinning is contiguous to the north wall of the sewer.

The 7-ft. circular sewer in Mott St. has a concrete cradle and a brick arch and an invert lining of vitrified brick. It was built entirely in open trench, which for about 970 ft. was very deep, the maximum depth being 35 ft. This required very heavy sheathing and bracing, and the work was further complicated by the presence of a 36-in. water main and a 4-ft. x 2-ft. 8-in. egg-shaped brick sewer within the lines of the excavation. The water main was supported by wire cable slings passed over heavy timbers spanning the trench and was blocked up on timber posts resting on the completed sewer before back-filling was begun; this prevents settlement and leakage as the back-fill settles. The invert of the brick sewer was above the top of the new sewer. The former was a part of the old system and the flow in it was northward or opposite to the direction of flow in the new sewer. The construction of the new sewer followed the excavation closely and, as the work progressed from Canal St. southward, the old sewer was destroyed in short sections just ahead of the new work and the flow, which was not large, was carried around the immediate site of construction work in wooden flumes and delivered to an 8-in. vitrified pipe sewer laid in the back-fill over the completed part of the new sewer. This pipe extended on a down grade northward to Canal St., where it discharged into the old system. House connections were provided on the 8-in. line, but these as well as the temporary sewer itself are to be abandoned when connections are made with the new sewers. The pipes for the new connections were built into the new masonry as it was laid and were carried up nearly to the surface of the ground and there boxed in till the final connections could be made. The excavation was carried on with pick and shovel and the spoil was hoisted and deposited as back-fill by a Lidgerwood cableway which also servied the Mott St. shaft as before stated.

The subway work in New York City and other work incidental thereto is done under the direction of the Public Service Commission, First District; Mr. Henry B. Seaman, chief engineer; Mr. George S. Rice, engineer of subway construction; Mr. Alfred Craven, deputy engineer of subway construction. All the sewer design and construction work carried on under the direction of the Commission is under the general supervision of Mr. Amos L. Schaeffer, division engineer of the Sewer Division; Mr. Thad. L. Wilson, assistant engineer, is directly in charge of this work. All the work described is included in a general contract for subway construction held by The Degnon Contracting Co. The sewer work, including the special underpinning, is being done under subcontracts by Mr. James Pilkington, Mr. Frank J. Conway, superintendent. The first shaft for the tunnel work was begun in November, and at the time these notes were prepared, late in February, the tunnel construction was practically finished.

Cast-Iron Strainers and weep-hole openings of special design have been used in the concrete slabs of solid ballast-floor bridges on the Memphis & State Line R. R., a cut-off of the Illinois Central R. R., around the city of Memphis, Tenn. The device is conical, 6 in. in diameter at the base of the inverted cone and 3 in. in diameter at the smaller end, where there is a tapped flange, to which a standard 3-in. flange is bolted for carrying the 3-in. down-spout. The upper part of the casting is recessed to form a seat for the perforated cover, which is set in the waterproofing material used for covering the concrete floor.

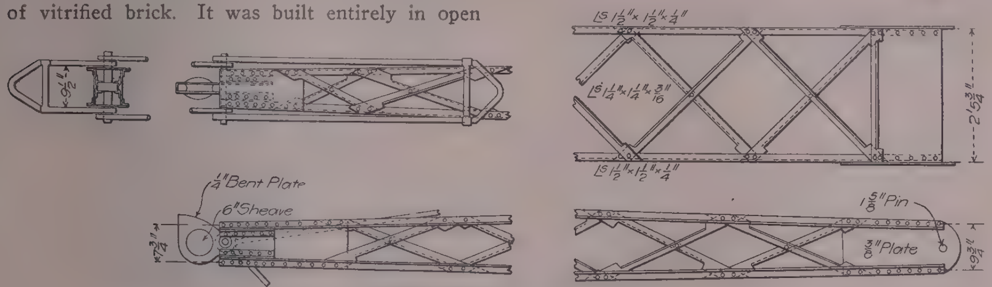
A Light, Long Steel Derrick Boom.

A steel derrick designed and constructed by the Chicago Bridge & Iron Works and used chiefly for the erection of structural steel work is of interest on account of its unusual reach and lightness and for the manner in which the lead lines are taken down through the hollow mast and arranged in such a way that they cannot become fouled in any position of the boom.

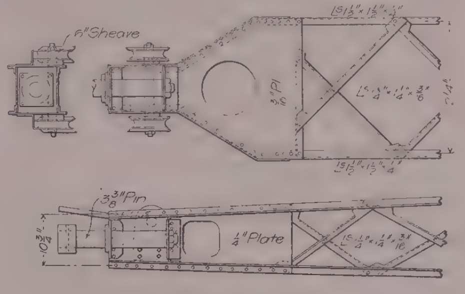
The mast and boom are each made in sections, the center one being interchangeable for either mast or boom. The end sections of the mast are about 24 and 25 ft. long and those of the boom are about 30 and 34 ft. long and the interchangeable middle section being 35 ft. long makes it possible to assemble the mast either 45 ft. or 80 ft. long and the boom either 67 ft. or 102 ft. long. This provides for a guy derrick with an 80-ft. mast and a 67-ft. boom or for a stiff-leg derrick having a 45-ft. mast and a boom either 67 ft. or 102 ft. long. The guy derrick has a capacity of 10 tons with the 67-ft. boom and the stiff-leg derrick has a capacity of only 5 tons for the 102-ft. boom.

All pieces of both mast and boom have rectangular cross sections made with four 1½ x 1½ x ¼-in. angle latticed on all four faces with single intersection 1¼ x 1¼ x 3/16-in. angles riveted together at their intersection. At the ends of the lattice bars the outstanding flange is cut and flattened into the planes of the other flange and is secured by two rivets which unite it to the main angle and to the adjacent lattice bar. The bottom and intermediate sections of the mast are about 2 ft. 4 in. x 2 ft. 6 in. throughout on center lines of the corner angle, all of the other members being tapered. The upper section of the mast is 2 ft. 4 in. wide throughout but tapers from 2 ft. 6 in. to 10¾ in. in the opposite direction. The boom tapers from 2 ft. 4 in. x 2 ft. 6 in. at the middle to 9½ x 7¾ in. at the outer end and to 2 ft. 4 in. x 9¾ in. at the foot. Each main angle has a splice 13½ in. long made with a 1¼ x 1¼-in. inside and a 1¾ x 1¾-in. outside cover splice angle shop-riveted to one member and bolted to the other with ½-in. bolts.

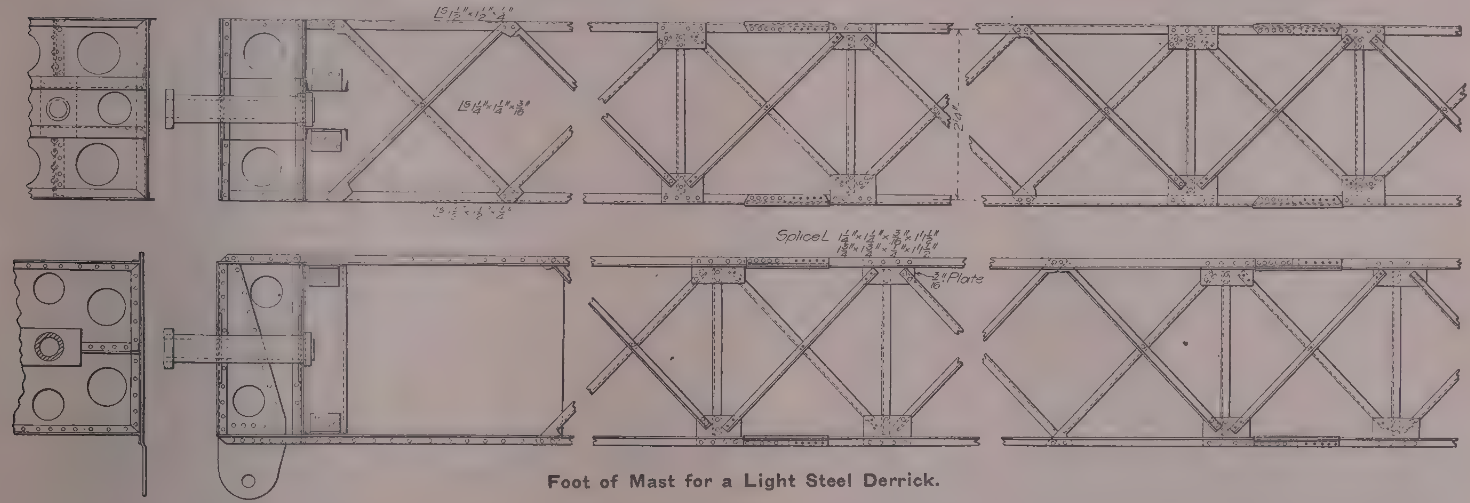
There are no web or flange cover plates except at the ends of the mast and of the boom.



Details of Steel Derrick Boom.



Top of Mast.



Foot of Mast for a Light Steel Derrick.

At the bottom of the mast there are two horizontal diaphragms which provide bearing for the 4½-in. hollow vertical pivot pin. Side plates riveted across opposite faces of the mast project beyond it and from jaws to receive the 1⅝-in. pivot pin at the foot of the boom. Wide cover plates on opposite sides of the mast afford bearing for inside hand crabs, not shown in the drawing.

At the upper ends of the mast the two parallel sides converge abruptly at the extremity and have bent cover plates, which, with the cover plates on the other two sides form a closed rectangular cross section providing a sort of projecting shoulder on each side of which there is an exterior sheave mounted on a common axle which passes through the mast. The four lines, from both tackles, lead over the sheaves, pass through holes cut in the inclined cover plate and through the hollow interior of the mast to the foot where they may be wound on the crab drum or continue through the hollow pivot to an exterior hoisting engine. The axle of the top sheaves also engages the ends of a U-shaped link to which it shackles the topping lift.

The side cover plates at the foot of the boom are bored to engage the pivot through the foot of the mast.

A 2⅛-in. horizontal pin through the outer end of the boom engages two U-shape links shackled to the hoisting and topping lift tackles respectively besides which a bent plate projecting beyond the extremity serves as a housing for a 6-in. sheave for a runner line or for any other purpose.

The weight of this mast and boom is very small and the section lengths are such that it can all be loaded on a single car, thus making it very convenient for shipment and erection. This derrick has been very satisfactory for light work where it has been used both with hand and steam power. It was designed and built by the Chicago Bridge & Iron Works and has been used by them for the erection of a number of steel buildings and towers as well as for a number of elevated water tanks.

Mr. Horton writes concerning it:

"I am inclined to think your description is more elaborate than the size of the derrick justifies. It was designed at the earnest solicitation of a certain foreman of the Chicago Bridge & Iron Works. He was going out to erect a tank for the General Electric Company at Schenectady, N. Y. Said tank was 60 ft. in diameter and 50 ft. high, the foreman voicing the suggestion that such a derrick would work very handily first standing alongside and placing the plates of the bottom, saving all awkward handling. When the bottom was riveted and down in place, move the derrick to the center of the tank. Before this particular work was completed we had contracted a tank for the city of Schenectady 90 ft. in diameter and 50 ft. high, which was manipulated in the same way with some raising of the derrick in its position in the center of the larger tank. However, in either case there was but one move made of the derrick. We have since then found it very useful, in fact have built others quite of the same design of somewhere three times the capacity. These and even the light one are useful in field work."

Oiling Bronze Plungers of large outside-packed plunger pumps such as are used for elevator or other high-pressure service, has been found in recent practice at a large industrial power plant at Baltimore, Md., to be of advantage not only in lessening the usual surface wear on the plungers from the packing, but also, by the material reduction of friction load, in increasing the sensitiveness in response to the automatic governor. The oil is fed to the top side of each pump plunger by a simple oil cup set for a very slow feed.

Water Softening at Oberlin, Ohio.

By W. B. Gerrish, Engineer and Superintendent.

It is now four years since the method of softening the city water with lime and soda ash was begun and sufficient time has passed to determine its real merit. As was to be expected, when the treated water was first used there was a vast amount of bitter opposition. There seemed to be no end to the tastes, smells and ill effects of the water, but in time the prejudice wore off and the ills disappeared. At present, if for any reason the water is not kept to the highest point of softness it is at once noticed and the attention of the superintendent called to the fact. It is doubtless true that the water is much better now than when we first began to treat, for experience has taught us many things and the methods of control and application of chemicals used at first were long since abandoned.

It was always considered one of the first principles of water softening that every particle of raw water should have its proportionate amount of chemicals mixed with it; and we took great pains to attain that end. The treated water under this method always contained at least 17 parts per million of residual caustic alkalinity and we supposed that that could not be avoided. We discovered, however, that if the chemicals for a day's run were introduced into one-half the raw water and the balance let into the basin without chemicals the caustic alkalinity disappeared. This was true until the colder weather came and the temperature of the raw water went below 13 deg. Cent., when the caustic alkalinity again appeared. We then began to add a small amount of sulphate of iron to the water as it flowed from the first basin to the second. The amount of iron was gradually increased until when the water reached its maximum density we were using 2 grains of iron per gallon. The iron sulphate removes all the caustic alkalinity and the water is as nice as in summer.

At first the amount of chemicals was determined empirically, but for a year we have analyzed the raw water for alkalinity, incrustants, magnesium and free carbon dioxide according to the methods recommended by the Laboratory Section of the American Public Health Association and then calculated the chemicals to be added. By this method much better and more uniform results have been obtained.

During the first part of 1907 we used a lump lime containing 89 per cent CaO and the rest of the year a hydrated lime containing 69 per cent CaO. Our lime bill could be reduced 25 per cent by buying lump lime in bulk. As it is the lime used during the year cost $521 and the soda $395, making the cost per 1000 gal. of water treated 1.3 cents.

It is of interest to note that during the whole summer of 1907 the raw water was alkaline to phenolphthalein, sometimes to the extent of 9 parts per million. This, however, did not interfere with the softening process.

During 1907, records were kept of the temperature of the raw water and also of the water in the pipe system, with the following results in centigrade readings: The raw water was at its maximum density or below with the water in the system 6 deg. until the middle of March. By the middle of May all the water was 13 deg. The maximum temperature of 27½ deg. for raw water was reached on June 25 and again on July 25, with the water in the mains 21 deg. and 22 deg. The temperature was above 20 deg. until the last of September and reached 13 deg. the middle of October. The maximum density was again reached the last of November.

The amount of lime used during 1907 varied from 3.4 to 9.4 grains per gallon; while the soda ash varied from 1.2 to 3.1 grains per gallon.

Chara has grown in the reservoir ever since it was built and at certain seasons of the year has given an offensive taste to the water. But little attention was paid to it until we began treating the water, when the taste was attributed to the treatment. Various attempts were made to kill the chara with copper sulphate without success. The Board finally decided to put in a float intake, as the surface water seemed to be always free from the taste. In the spring of 1906 the storage reservoir was drawn down about 6 ft. and a new intake laid, provision being made for taking off the float in winter and allowing the intake pipe to rest on a support about 2 ft. below the surface. All the chara exposed by the lowering of the water was carefully removed.

The following summer was dry and the reservoir was not filled until winter, so that the area where most of the chara grew was exposed to the sun for the whole summer. In 1907, the growth of the chara was more luxuriant than ever before, but in spite of this fact there was no time during the year when the ordinary person could detect any taste. An expert visiting the plant in the summer said he could detect a slight taste.

This fall the float was not removed when the first thin ice had formed. One day we discovered that the basins were not filling as rapidly as they should and that the water was overcharged with chemicals. Investigation showed that the muskrats had carried grass to the mouth of the intake. The grass was washed along to the fish screen where it collected and almost cut off the flow of raw water. The fish screen has now been placed at the end of the intake.

In previous reports we have noted the tendency of the sand in the filters to become coated with lime. This process kept up until last spring when we found that the filters were not doing any work. Upon removing the sand it developed that the sand in the upper part of the filters was heavily coated, while the lower part was cemented into a nearly solid mass. How the water got through seemed a wonder. After cleaning the strainers with hydrochloric acid the filters were renewed with lake sand having an effective size of °.22 m.m. and a uniformity coefficient of 1.5. This was of course a very fine sand, but the idea was that as the grains became coated with lime they would increase in size and so become better with age. The filters worked all right for a few months and then the loss in head gradually increased until it was too great to make the further use of the filters practicable. When this sand was moved we found that it contained so much soft material that the attrition of washing and use had broken it down to a mud on the surface of the filter. We then put in Cape May sand as originally used. It is hoped, now that we have a water free from caustic alkalinity, that the formation will be much slower than formerly.

The use of a lime-treated water without recarbonating has been regarded as an experiment from the first and a close watch has been kept on all places where the water goes to see that coatings were not forming which in time would do harm. The pipe system seems to be free from any coating and the only place where a deposit has formed has been the inside of one make of rotary meters. Of 70 disc meters but one required cleaning on account of lime deposit. Of 270 rotary meters of one make none required cleaning, while of 390 of another make 10 per cent required cleaning. Some of these, however, had been in use for ten years or more without touching, so they needed cleaning on general principles.

There are hard deposits from the water during the process of treatment in the mixing box and the pipe leading from the same. The mixing box has to be opened and cleaned once a year and the pipe leading from the box twice a year. The sludge formed from the precipitate is turned into a near-by creek every spring and fall when there is sufficient water running to properly dilute it and carry it off.

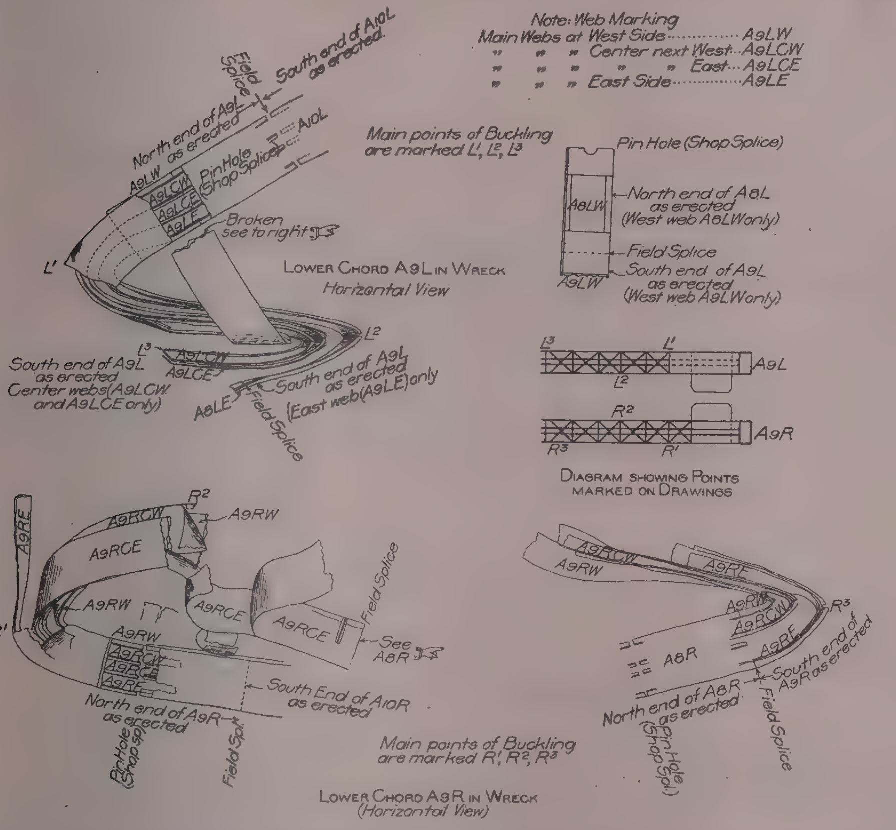
APPENDIX 15, REPORT OF ROYAL COMMISSION ON QUEBEC BRIDGE.

A Description of the Various Experimental Researches That Have Been Made in Connection with the Building of the Quebec Bridge and During This Enquiry.

[Slightly Condensed.]

The Phœnix Iron Company possesses the most powerful machine for compression tests in existence; unfortunately, some doubt exists as to the accuracy of the records obtained from it. As a result of a series of tests made in 1897, the New York Department of Buildings places its error in compression at 15 per cent. in excess; in tension, however, its results seem to be in agreement with those obtained with other machines on similar material. In spite of these doubts, however, this machine has been of invaluable service to the engineering profession.

In evidence Mr. David Reeves, the president of the Phœnix Bridge Co., states that he had given orders "that all the special tests advised by the consulting engineer, Mr. Cooper, or by our own engineers, arising from the unusual size of the bridge be promptly and fully made," so that from the outset the designers of the Quebec bridge had at their disposal both the equipment for making tests and the authority to use that equipment.


Note: Web Marking
Main Webs at West Side A9LW
" " " Center next West .. A9LCW
" " " " " East .. A9LCE
" " " East Side A9LE
Main points of Buckling are marked L¹, L², L³
South end of A10L as erected
Field Splice
North end of A9L as erected
A10L
Pin Hole (Shop Splice)
A9LW
A9LCW
A9LCE
A9LE
Broken see to right
L¹
L²
L³
Lower Chord A9L in Wreck Horizontal View
South end of A9L as erected Center webs (A9LCW and A9LCE only)
South end of A9L as erected (East web (A9LE) only)
Field Splice
Pin Hole (Shop Splice)
North end of A8L as erected (West web A8LW only)
A8LW
Field Splice
South end of A9L as erected (West web A9LW only)
A9L
A9R
R¹
R²
R³
Diagram showing Points marked on Drawings
A9RE
A9RCW
A9RW
A9RCE
See A8R
South End of A10R as erected
North end of A9R as erected
Pin Hole (Shop Spl.)
Field Spl.
Main points of Buckling are marked R¹, R², R³
Lower Chord A9R in Wreck (Horizontal View)
A8R
North end of A8R as erected
South end of A9R as erected
Field Splice

Drawing 19. Report of Quebec Bridge Commission. Approximate Horizontal Projections of Wrecked Lower Chords A9L, A9R.

The evidence shows that along certain lines these facilities were by no means neglected, and we are of opinion that had Mr. Cooper and Mr. Szlapka realized how limited is our knowledge of the strength of compression members, they would have made as much use of the testing machine for compression tests as they did in connection with the eyebars.

The appliances and tackle that were so successfully used in the erection were tested when necessary, and the only failure of which we have record was in the hook that was lifting A-9-L in the Chaudiere yard.

Some of the tests which were made are as follows: Two plates about 28 x 2⅛ in. in section were tested in tension to determine the efficiency of the connection between the two pins to be used at certain intersections. The plates were tested with 12-in. pins and reinforced bearings; the records filed are rather meagre. One plate dished at one pin bearing when the stress amounted to 35,200 lb. per square inch, the test having continued without sign of failure to 26,000 lb. per square inch. In the test of the second plate the rivets began to work loose at a stress of 16,000 lb. per square inch; the test was discontinued before failure, when a stress of 26,000 lb. was reached.

An eyebar 16 x 1¾ in. was made into two by cutting and reheading. One half was bent into a long "S," the maximum deviations from the line between centers of pin holes being about 3½ and 4⅜ in.; the length center to center of pin holes was about 17 ft. The other half was tested to destruction as a straight bar and failed under a stress of 57,990 lb. per square inch; 14-in. pins were used. The bent half stood a stress of 61,340 lb. per square inch before it failed. The bends were made in the plane perpendicular to the pins. The test was assumed to indicate the negligible effect of waves and bends in tension members.

Mr. Cooper having questioned the efficiency of the device for adjusting the position of the suspended span on account of the friction between the pins and the toggle eyebars, tests were made to determine the correctness of his opinion. The tests were not conclusive and Mr. Cooper decided that some entirely different device should be used at the north end of the suspended span.

An important series of tests was made at Mr. Cooper's direction upon the deformation of eyebars under strain. The usual record of tests upon full-sized eyebars will be found in exhibit No. 88 [omitted]; 73 tests in all were made. Squares were scribed on the heads of a number of these and observations were made both of the flow of the metal near the eye and of the deformation at the pin-bearing. This study has been fully described by Mr. Cooper in his paper entitled "New Facts about Eyebars," presented at the meeting of the American Society of Civil Engineers, March 21st, 1906.

Alterations were made in the dimensions of the eyebar heads as a result of these tests and the set at the pin bearing was allowed for in the camber diagrams. The above were all of the special tests made in connection with the design of the bridge.

After the collapse of the bridge, the Phœnix Bridge Co., at its own cost, and on its own initiative, built and tested the chord shown on drawing No. 22. This model chord had, as far as possible, the same relative dimensions as the No. 9 chords of the Quebec bridge and yet was small enough to be broken in the Phœnix Iron Co.'s testing machine. The test was made on November 21 and 22, 1907, and was under the general direction of Prof. W. H. Burr. By the courtesy of the Phœnix Bridge Co. we are able to give the text of Professor Burr's report:

"New York, December 23, 1907.

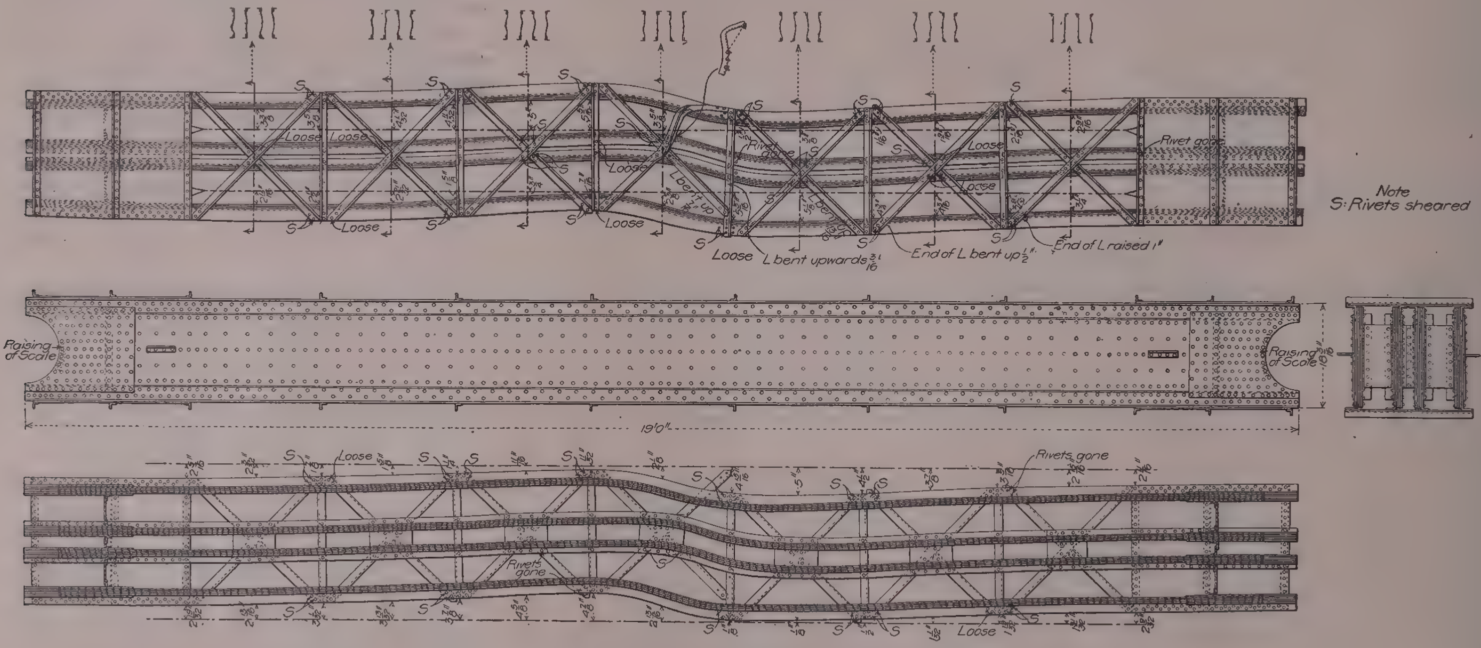
"Mr. David Reeves, President,
"The Phœnix Bridge Company,
"Philadelphia, Pa.

"Dear Sir—In accordance with your instructions a model chord section was built to a linear scale of one-third of the lower chord section 9 of the anchor arm truss of the Quebec bridge and was tested to destruction, under my direction and supervision, at the shops of the Phœnix Bridge Company at Phœnixville on November 21st and 22d of the current year. The purpose of this test was to secure all possible information regarding the circumstances and method or other features of the failure of that chord which could be disclosed by the test of the model column in question.

"This chord section was built of four ribs 54 inches deep, with 4 x 3 x ⅜-in. double angle latticing. Its area of cross section was 780 sq. in.

"All the linear dimensions of the model were exactly one-third of those of the full size chord section, making the area of cross section (86,526 sq. in.) one-ninth of that of the full size member and the volume of metal, with the exception about to be noted, one-twenty-seventh of the original member. This exception arises from the fact that the actual chord member as built, 57 ft. 9/32 in. in length, had a heavy chord joint in it 10 ft. 6 in., a little more than twice the depth, from one end. Furthermore, the full-size chords were bored for 12-in. pins, and pins of the same diameter were used for the end bearings of the model chord section. It is manifestly impossible to reproduce in a test precisely the conditions existing in the structure at the time of its failure, but it is believed that the end conditions employed in the test and the accurate reproduction by scale of the main dimensions and nearly all the dimensions of the details in the model enable the nearest approach to the actual conditions of the structure to be secured. It is believed that these unavoidable and subordinate departures from the actual conditions of the chord member did not sensibly affect the conditions of failure in the testing machine or the ultimate load carried by the model.

"The blue print plans accompanying this report show both the working drawings of the original chord members 8 and 9, including the joint mentioned above, and those of the model chord precisely as it was built as well as in its condition after test, the latter plan having been made from accurate measurements of the failed member immediately after its removal from the testing machine. The blue prints of the model show the four webs of the original chord accurately reproduced by scale, making the depth 18 3/16 in. and the length 19 ft. As the plans of both the actual chord and the model show every main and detailed dimension it is not necessary to repeat them here. It should be stated, however, that each of the two interior ribs was composed of one 18 x 5/16-in. plate, one 18 x ¼-in. plate, two 15 5/16 x 5/16-in. side plates, and two 2 11/16 x 1¾ x 5/16-in. angles; and that the two exterior ribs were each composed of one 18 x 5/16-in. plate, two 18 x ¼-in. plates, one 12⅝ x 5/16-in. side plate, and two 2 11/16 x 2 x 5/16-in. angles. The latticing was a double oblique system of 1 11/32 x 1 x ⅛-in. angles with 1 3/16 x 1 x ⅛-in. crossing angles at the panel points of the former at right angles to the axis of the member. All of these lattice angles had two 7/24-in. rivets at the ends of each with a single rivet at each crossing of the interior flange angles of rims, as clearly shown on the plans. The linear scale of one-third of the actual dimensions required the rivets used to be ¼, 5/16 and 7/24-in. in diameter, also as shown on the plans, the 7/24-in. rivets being turned down from an original diameter of 5/16 of an inch. Similarly the 2 11/16 x 2-in. and the 2 11/16 x 1¼-in. rib flange angles were planed down from 3 x 2 x 5/16-in. pieces. The lacing angles were also planed from 1½ x 1⅛ x 1⅛-in. and 5/32-in. angle to the dimensions given above. All rivet holes were drilled.



Drawing 21. Report of Quebec Bridge Commission. Results of Tests of Model Chord 1.

"The method of construction of the model was such as to leave it in true and accurate condition. The web plates were laid off by wood templet, except the pin-plate holes, and drilled while the pin plates were drilled from iron templets. The pin plates were then used as templets for the drilled holes at the ends of the web plates. One web plate for each rib thus drilled was used as a drilling templet for the other plates of the same rib, the blank plates being bolted to the drilled plate for this purpose. In the same manner the blank flange angles were bolted to the drilled webs and drilled from the latter as a templet. Rivet holes required for lattice angles were drilled from iron templets, but the batten plates first drilled were used for drilling templets after the chord was completely assembled. After the component parts of the ribs were drilled they were taken apart, cleaned, painted and bolted together for riveting. The latter was done both in web and lattice angles with pneumatic hammers. The lattice bars were drilled like the other parts of the model. After the riveting was completed the pin holes were bored and subsequently the ends were faced to proper dimensions in a rotary planer. All the metal used for the main parts of the model column was medium steel, but soft steel was used for rivets. The steel plates were furnished by the Lukens Iron & Steel Company, of Coatesville, Pa.; but the angles were supplied and rolled by the Phœnix Iron Company, of Phœnixville, Pa. The rivets were purchased in Philadelphia.

"In order that the character of the metal employed might be completely determined, tensile tests were made of both plates and angles and shearing tests of both the 7/24-in. rivets used in the model and ⅞-in. rivets in the full size chord.

"The following tabular statements show the results of all these tests and of representative specimen tests of the metal used in the chord member 9 as actually built, together with chemical analyses exhibiting the main elements of interest in such structural material:

Tensile Tests of Plates and Angles.

Date	Heat No. or Size	Pounds per sq. in. Elas. Lim.	Ultimate.	Per cent St'gth in 8"	Reduction.	Fracture.
1½" x 1½" x ⅛" Angles.						
Nov. 6	1402	52,520	65,660	27.	55.6	Silky
" "	"	50,000	63,460	23.	57.7	"
" "	"	51,900	62,500	27.5	59.6	"
" "	"	50,340	61,300	21.	54.4	Silky cup
" "	"	50,360	65,700	20.5	48.2	" "
3" x 2" x 5/16" Angles.						
Nov. 5	1492	43,300	63,040	31.25	61.3	Silky
" 6	"	41,780	62,100	32.	64.	"
5/16" Plates; test specimens 1.045 inch wide.						
Oct. 29	13673	38,370	65,420	29.	53.1	——
" "	"	37,350	64,200	30.	53.1	——
Plates of chord member 9 as built.						
1904						
Nov. 1	54½x⅛	36.840	60,680	26.5	53.	Silky cup
" "	54½x3/16	40,810	61,440	25.5	51.4	" ang.
" "	" "	42,000	67,700	23.	60.5	" "
" "	" "	40,780	65,540	24.5	49.	" cup
Angles of chord member as built.						
1904						
Sept. 14	8x8½x15/16	38,000	61,000	27.	52.3	Silky cup
" "	8x8" x15/16	37,150	63,920	29.	50.8	" "
" "	8x6 x15/16	39,460	62,800	30.	47.1	" "
" "	"	38,890	61,300	32.5	49.2	" "
" 30	4x3 x⅜	41,730	67,640	29.5	.26	" ang.
Nov. 18	"	42,710	64,860	27.	.26	" "

Shear Tests on Rivets—Nov., 1907.

Size of rivets.	Ultimate resist in lbs. per sq. in. average.			
7/24 in. diam....	59,700	58,200	59,700	59,200
⅞ in. diam......	50,420	50,875	51,380	50,960

Chemical Analyses.

	Car.	Phos.	Man.	Sul.
1½"x1½"x⅛" angles	.16	.038	.51	.037
5/16" plates	.21	.016	.40	.023
" "	.23	.025	.42	.024
54½x⅞ plates	.17	.01	.46	——
54½x13/16 plates	.17	.01	.46	——
" " "	.26	.007	.34	——
" " "	.26	.007	.34	——
8x3½x15/16 angles	.16	.041	.36	——
8x3½x15/16 "	.16	.041	.36	——
8x6x15/16 "	.17	.052	.39	——
" " "	.17	.052	.39	——
4x3x⅜ angles	.18	.036	.66	——
" "	.19	.05	.41	——

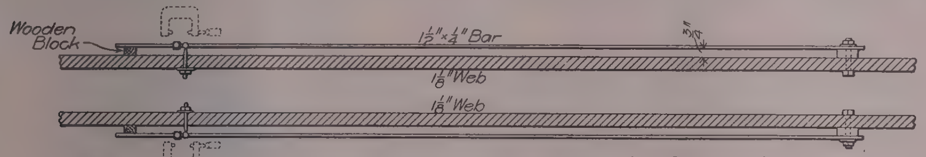
"The specimen tests of the plates and angles used in the actual chords were selected by me out of a large number so as to give a reasonable and comprehensive view of all and they are fairly representative. It will be observed that the usual effect of rolling thin metal necessarily finishing at a lower temperature than that in the heavier sections is apparent in the high elastic limits of 1½ x 1½ x ⅛-in. angles. The same effect, but to a smaller degree only, is probably discoverable in the 5/16-in. and ⅜-in. angles. This marked effect in the lattice angles of the model column has a distinct bearing upon the final results of the test. A similar general observation, and to a marked degree, applies to the higher unit shearing values of the 7/24-in. rivets as compared with those of the ⅞-in. rivets of the full size member.

"After placing the model column in the machine and under a load producing a stress of 12,000 lb. per square inch it was thought that a buckling or bulging of the web plates was discovered to the extent of .034 in. near the west end of the north rib, but this was found not to increase under further loading. Although measurements of this particular feature were not made before placing the column in the testing machine, continued observations subsequent to the first indicate, I think, conclusively that this particular deformation existed in the column before loading and hence that it had no effect upon the ultimate failure of the column; or, in other words, that it was an unavoidable result of the processes employed in the manufacture of the column and was not a true buckling of the plates under loading.

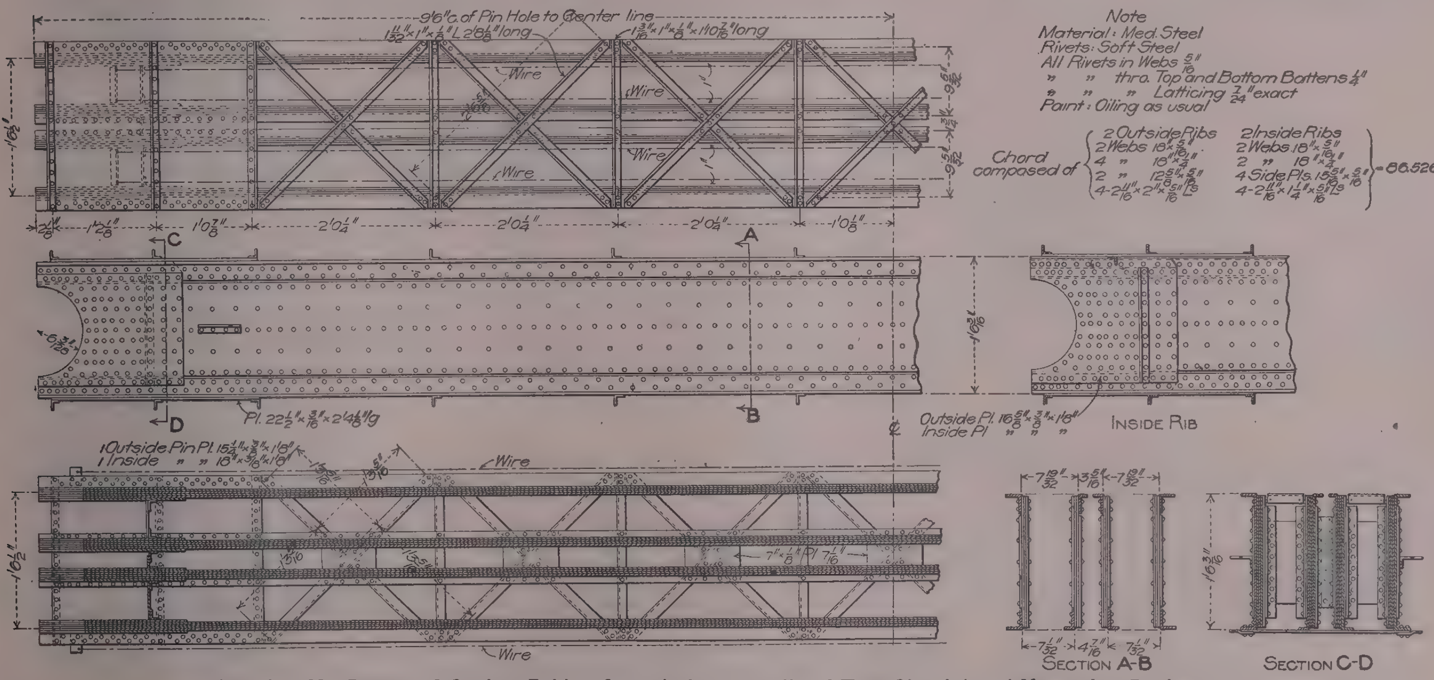
"The column was accurately placed in the machine with four fine wires stretched throughout its length in the general plane of the upper flanges and with two similarly placed in relation to the lower flanges. These fine wires stretched with constant weights enabled any vertical or horizontal deflections of the tops of the four ribs and the bottoms of the two exterior ribs to be measured by the aid of finally graduated steel scales. Furthermore, longitudinal timber scantlings on the two center lines of the exterior ribs, carrying steel scales at their ends, were used to measure the shortening of the column under loading for 16 ft. of its length to 1/128 of an inch. While these methods of measurements were not so refined as it might be desirable to adopt in an extended series of tests of this nature, they answered well the purposes of this particular investigation which was not intended so much to determine with refined accuracy all the deformations produced in the test as to discover the main features and methods or other circumstances of failure, so far as possible, which attended the collapse of the full size chord section.

"Prick punch marks were made in the heads of the rivets of the lattice bars throughout the length of the upper side of the column as it lay in the testing machine, and the distances between these marks were accurately measured at all stages of the test up to failure in order to ascertain the condition of stress in the lattice angles under the progressive loading to which the column was subjected. Furthermore, these bars were tapped with a hammer at the same time in order to secure further information as to their condition of stress as the tone of the resulting sounds might give.

"The progressive loading was applied in stages of 3,000 lb. per square inch of cross section of column, beginning with an initial loading of that value. At the end of every other stage of each loading the column was relieved of stress in order to make observations in that condition. This programme was adhered to up to a stress of 21,000 lb. per square inch, when the next increment was made 1,500 lb. per square inch, after which the column was freed of load. The remaining programme of loading is shown on the blue print plan showing the effect of the test on the column, which will be discussed in full below.



Micrometer Arrangement for Measuring Compression.


9'6" c. of Pin Hole to Center line
$1\frac{11}{32}$"×1"×$\frac{1}{8}$" L $2'8\frac{1}{8}$" long
$1\frac{3}{16}$"×1"×$\frac{1}{8}$"×$1'10\frac{7}{16}$" long
Wire
Wire
Wire
Wire
C
D
A
B
$1'0\frac{7}{8}$"
$2'0\frac{1}{4}$"
$2'0\frac{1}{4}$"
$2'0\frac{1}{4}$"
$1'0\frac{5}{8}$"
Pl. $22\frac{1}{2}$"×$\frac{3}{16}$"×$2'4\frac{5}{8}$" lg
Outside Pl. $16\frac{3}{8}$"×$\frac{3}{8}$"×$1'6$"
Inside Pl " " "
Inside Rib
1 Outside Pin Pl. $15\frac{1}{2}$"×$\frac{3}{8}$"×$1'6$"
1 Inside " " 16"×$\frac{3}{16}$"×$1'6$"
Wire
Wire
Section A-B
Section C-D
Note
Material: Med. Steel
Rivets: Soft Steel
All Rivets in Webs $\frac{5}{16}$"
" " thro. Top and Bottom Battens $\frac{1}{4}$"
" " " Latticing $\frac{7}{24}$" exact
Paint: Oiling as usual
Chord composed of
2 Outside Ribs
2 Webs 18"×$\frac{5}{16}$"
4 " 18"×$\frac{1}{4}$"
2 " $12\frac{5}{8}$"×$\frac{5}{16}$"
4-$2\frac{11}{16}$"×2"×$\frac{5}{16}$" Ls
2 Inside Ribs
2 Webs 18"×$\frac{5}{16}$"
2 " 18"×$\frac{1}{4}$"
4 Side Pls. $15\frac{5}{16}$"×$\frac{5}{16}$"
4-$2\frac{11}{16}$"×$1\frac{1}{4}$"×$\frac{5}{16}$" Ls
= 86.526□"

Drawing 23. Report of Quebec Bridge Commission. Details of Test Chord 1 and Measuring Device.

"After the application of each 3,000 lb., or finally 1,500 lb., increment of loading and upon each removal of loading an accurate series of measurements for shortening of the column, for horizontal and vertical deflections at the various panel points of the latticing and for the stretching or shortening of the latticing angles were made. As the column deflections were small the methods of measuring them were not altogether conclusive as to their amounts or as to their actual existence in some cases. At 9,000 lb. per square inch, for instance, three ribs showed an apparent upward deflection of 1/32 in. at and in the vicinity of the center of the column. This deflection did not appear to increase until the stress reached 18,000 lb. per square inch, and then only to an amount less than 1/16 in. with doubt as to the accuracy of the measurement. No apparent increase of deflection was found again until a stress of 24,000 lb. per square inch was reached, when the deflection of the four ribs appears to be 3/32 in., 1/32 in., 3/32 in. and ⅛ in. respectively at center. On removal of the load this deflection disappeared entirely except for 1/32 in. in one interior rib and the same amount in one exterior rib, both being upward. There was no subsequent opportunity to make further deflection measurements.

"Under a stress of 12,000 lb. per square inch one rivet in a lattice angle at the second center intersection from the west end of the column was found loose, but toward the end of the test it appeared to become less so, the conditions of the center ribs probably becoming such as to give it less opportunity for small motion. Up to the final loading all other rivets appeared to remain in good condition, although they were frequently tested with a light hammer.

"Actual testing of the column with the application of the first loading began at about two o'clock in the afternoon of November 21st of the current year and it was continued without interruption in the manner set forth in the preceding statements to 11 P. M. of the same day. At that time a load of 25,000 lb. per square inch was reached for a very short time in the endeavor to attain a stress of 25,500 lb. per square inch. This endeavor, however, was unsuccessful in consequence of the leaking of a pump valve (subsequently repaired) to such an extent as to render it impossible to secure the desired pressure in the cylinder of the testing machine.

"After having attained the above loading of 25,000 lb. per square inch the programme of the test was interrupted until 10 A. M. of November 22d.

"At that time instructions were given to load the column to 25,500 lb. per square inch, but inadvertence in signaling to attendants at the pump caused the load to reach 26,850 lb. per square inch, at which stress the member suddenly failed. This failure was attended by a quick, sharp report and it occurred so suddenly that three observers who were closely watching the column at the time could not discover any sequence in the yielding of the details of the column; the occurrence was so sudden that all failures of details appeared to be absolutely simultaneous.

"Aside from the raising of scale on the pin plates immediately in front of the 12-in. pins, the collapse of the column consisted in the failure by shearing of the majority of the lattice rivets at the central panel of latticing and of a considerable number of other rivets throughout the length of the column in both flanges, leading to the permanent bending to reversed curvature of the four ribs at the same central vicinity accompanied by the violent bending or distortion of the lattice angles and some small dishing of the rib web plates, all as shown on the accompanying blue print. The ribs were all slightly bent immediately beyond the supporting influence of the battens at each end.

"There are certain features of this practically instantaneous failure of the column which are highly significent. As indicated in the preceding statements, there were no permanent strains or distortions of any kind discovered or apparently discoverable up to the loading producing failure. This observation is certainly true of every part of the column except the 7/24-in. lattice rivets. If suitable apparatus for refined measurements could have been applied to them some shear distortion might have been observed prior to the final loading. Observations made on the latticed angles showed no permanent stretching or compression of those members prior to failure. The phenomenally high elastic limit of the metal in them shown by the test results in the tabular statement and already remarked upon indicates that they would have exhibited no marked permanent distortion much short of ultimate resistance either as tension or compression members. In point of fact all the circumstances of the test indicated that no main part of the column was stressed up to its elastic limit; in other words, that the entire loading was insufficient to develop more than a part of the elastic resistance of the column as a whole and that if the latticing details had been stronger the column would have carried a greater load before collapsing. The instantaneous failure was clearly due to the fact that the main parts of the column were subject to elastic stress only.

"Although it is impossible to correlate accurately the results of this column test with the stress conditions in the actual chord section at the instant of failure, in consequence of the higher elastic qualities of the relatively thin metal in the model column which has greatly been commented upon and the presumably greater care which is usually bestowed upon the manufacture of a model member, an approximation of some value may perhaps be made.

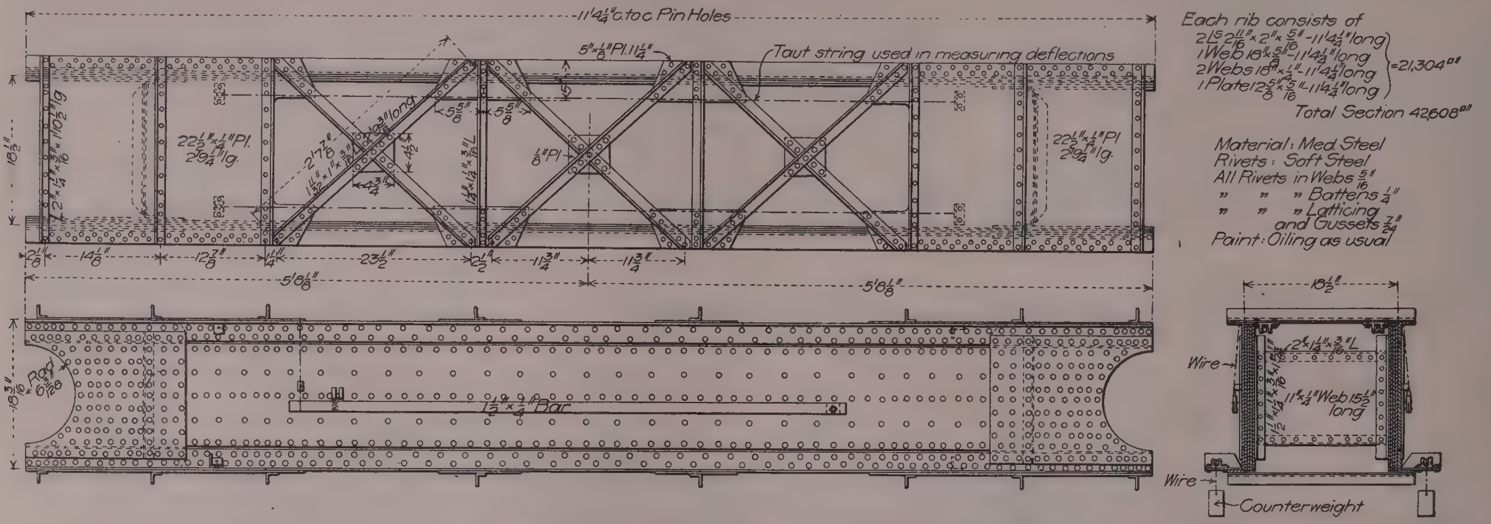
"The friction of a hydraulic testing machine is known to be considerable, but without recent calibration its amount cannot be confidently stated. The friction of the machine at the Phœnix Bridge Company's works was determined some ten or twelve years ago by G. Henning as 17½ per cent. of the total load on the piston as indicated by the mercury gauge, and this may be accepted provisionally until a further calibration can be made. If this percentage deduction be made from 26,800 lb. per square inch, the apparent stress at which the column failed, it will make the compressive stress in the metal 22,110 lb. per square inch. The sheer tests of the 7/24-in. and ⅞-in. rivets make the average of the latter but 86 per cent. of the former. Hence, if the ultimate shearing resistance of the 7/24-in. rivets had been the same as that of the ⅞-in. rivets, the stress on the column producing the failure of the latticing rivets would have been but 19,014 lb. per square inch of the column. Just what value should be given to the possibly higher excellence of manufacture of the model over that of the full size column is of course not determinable. It may or may not have sensible value. It is to be noted, however, that after making such allowance as is practicable for the friction and the increased resistance of the smaller rivets there is reached an intensity of stress nearly identical with that which existed in the actual chord section at the time of this failure.

"It should be carefully observed that the radius of gyration of the normal section of the model column about an axis at right angles to the webs and through their centers, i. e., parallel to the axis of each pin, is 5.43 in. and 5.52 in. about a central axis parallel to the webs. Hence, as the column lay in the testing machine, the ratio of its length divided by the horizontal radius of gyration is 35, while the ratio of the same length over the vertical radius of gyration is 42. The column failed therefore in the plane of the greatest radius of gyration. Furthermore, its failure was wholly in a horizontal plane, there being no sensible vertical deflection of the failed column.

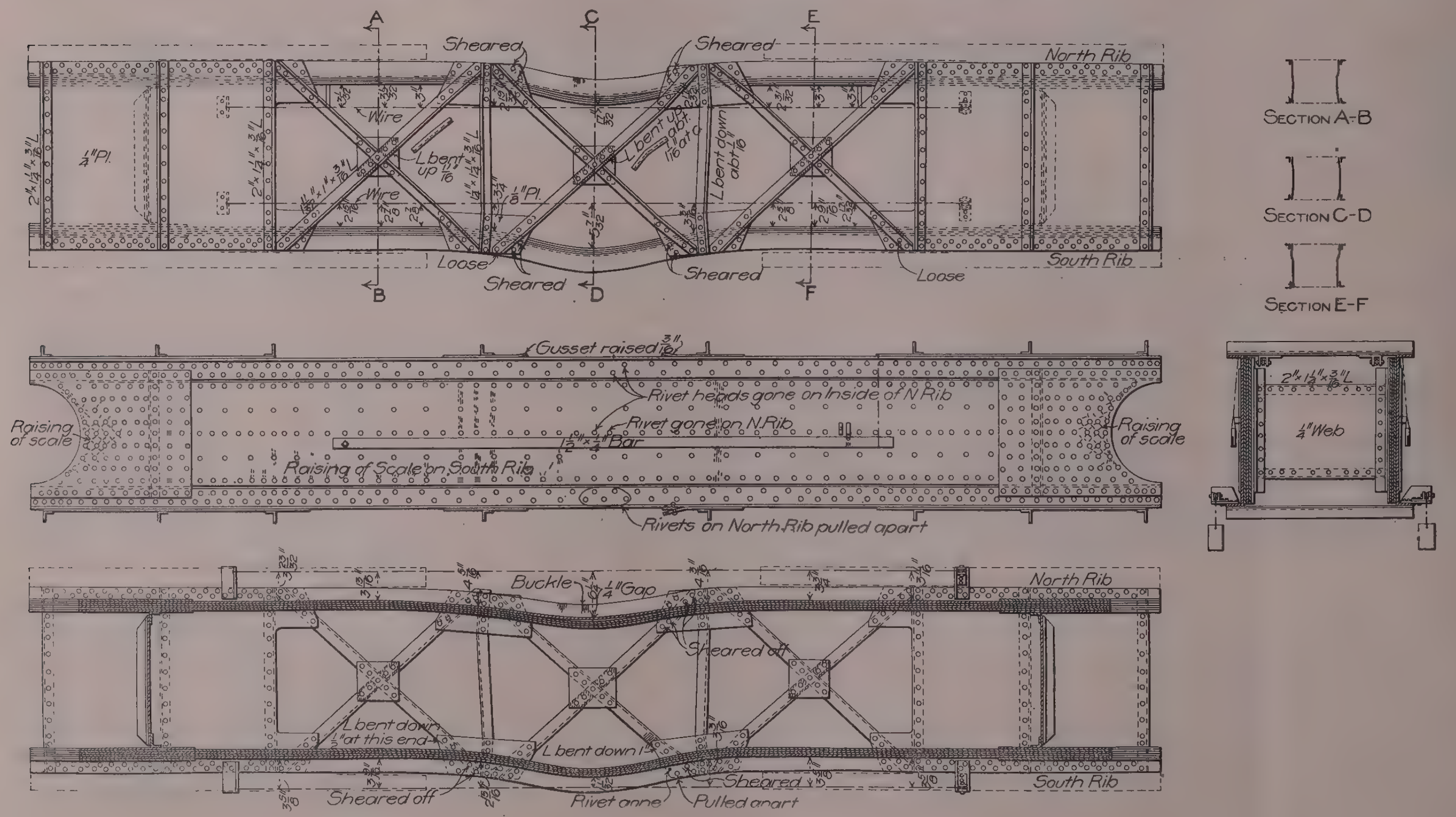
"The length of the column was such as to place it practically at the limit between short and long columns, as the ordinary column formulæ, such as the much used Gordon's and 'straight line,' are properly applicable when the ratio of length over radius of gyration has values greater than about 40 or possibly a little more. Inasmuch as the ultimate carrying capacity per square inch of section increases as the length of column decreases and as this model column was comparatively short, the latticing required to develop its full load carrying power should be relatively heavy rather than light.

"Very truly yours,
(Signed) "Wm. H. Burr,
"Cons. Engr."

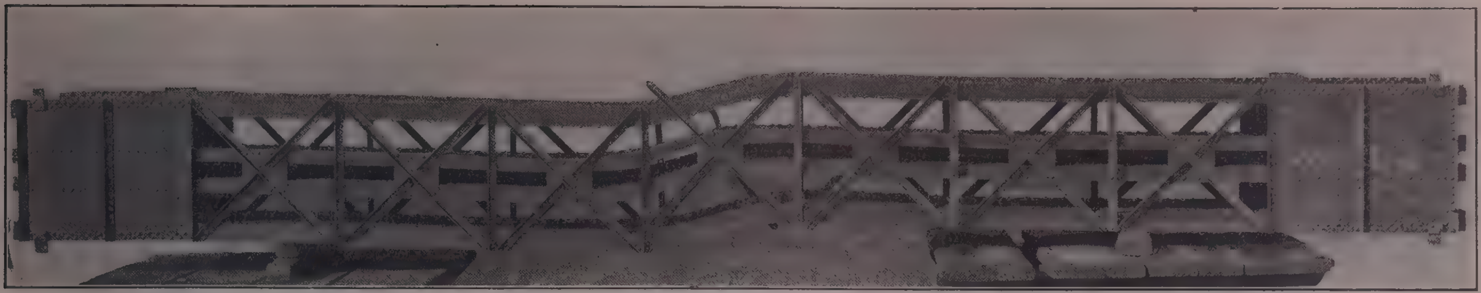
The Commissioners were invited to be present and to assist at this test, and the Department of Railways and Canals was represented at it by Mr. C. C. Schnieder. The shape of the model chord after the test was finished is shown on



Drawing 22. Report of Quebec Bridge Commission. Details of Test Chord 2.



Drawing 24. Report of Quebec Bridge Commission. Results of Tests of Model Chord 2.



Appearance of Model Chord 1 at the Conclusion of the Test.

drawing No. 21, which has been prepared from the blue prints referred to by Professor Burr. The accompanying photographs show the details of the failure very clearly.

The Commission has the following comments to make concerning this test:

(1) There was little or no indication of failure up to the instant at which it occurred. Failure took place with explosive violence, by the shearing of the outer rivets of the latticing.

(2) Messrs. Schneider, Deans and Szlapka were closely watching the chord when the unexpected failure occurred. No one of these engineers could say what connection or detail was the first to give way.

(3) It was noted that the surface scaled at only three outside pin bearings.

(4) The cross lattice bars showed little sign of stress, rivets being sheared only in those bounding the central bay. Theoretically, under a racking stress in the column, these bars would not come into play, the diagonals only being strained, one set in tension and the other set in compression.

(5) The failure of the riveting was systematic. In each lattice bay one diagonal connection failed in tension and the other in compression; and usually both failures occurred at the side of the bay farthest from the nearer end of the column. It will be noted from the photographs that both on top and bottom the diagonals in the center bay failed in the opposite manner to that of the corresponding diagonals in all the other bays.

(6) The efficiency of the central connection plate in the bottom latticing is well shown by the photographs.

(7) Some of the lattice rivets were cut out and found to be partially sheared and there were some slight indications that they had been sheared first on one side and then on the other, indicating a reversal of stress in the lattices. That such a reversal would instantly follow the failure of the latticing in the central panel is apparent from the curvature of the chord on one side of the central panel, being opposite to that on the other side.

(8) Subsequent investigation has shown that the method adopted for testing the working of the lattice bars was unsatisfactory. It consisted of measurements between center punch marks on the rivet heads and did not include the effect of rivet shear.

In our opinion the load was more evenly and centrally applied in this test than it would be in the case of a chord in ordinary service. In other words, under working conditions the failure of the latticing of this model chord would have taken place under a smaller stress.

This is the more probable since, as Professor Burr points out, the model chord was superior to the bridge chord in both material and workmanship. The difference in material is well shown by the test records included in Professor Burr's report.

On Nov. 26 some tests on rivet shear were made by the Phœnix Bridge Co. The results of these tests, together with those given by Professor Burr, showed that the rivets used in the bridge would develop an ultimate strength of slightly over 50,000 lb. per square inch, and also that the rivets maintained without failure their ultimate strength, even though partially sheared.

On Jan. 14 some further tests were made by the Phœnix Bridge Co. which gave similar results. A movement of from 3/16 in. to ¼ in. apparently took place before actual failure.

This rivet shear offered a reasonable explanation of part of the change in the lengths of the diagonal lattice bars which must have accompanied the distortions of the chords in the bridge which were measured on Aug. 27, 1907. The inspectors had indeed carefully examined the chords and the lattices and reported no evidence of failure, but this rivet shear might easily have escaped their observation; it is noteworthy that no one of the engineers assembled to watch the test of the model chord on Nov. 21 thought of it, and they completely failed to detect such action up to the moment of failure although working under the most favorable conditions. A change in length in addition to the above seemed to be due to the reduced section at the center of one of each pair of diagonal lattice bars.

In December the Commission ordered the construction of test chord No. 2 for the purpose of determining the strength of the webs of the design used in the Quebec Bridge. The dimensions of this chord are given in drawing No. 23. It had a section half that of test chord No. 1; the number of rivets in the lattice connections was doubled, the section of the lattice bars was increased 50 per cent. and the weak points at their centers were strengthened by the use of connection plates. The webs were of the same section as the outer webs of test chord No. 1. Material from the same heats was used in the manufacture of the two test chords.

This chord was tested at Phœnixville on Jan. 18, the observations made during the test and the shape of the chord after failure being shown on drawing No. 24 and on photographs.

It will be noted that this chord failed under a stress of 37,000 lb. per square inch by the buckling of the webs in the center bay, the latticing being sufficiently strong to fully develop the strength of the webs. The nominal strength of the column (the record has to be corrected for an unknown machine error) was slightly less than the elastic limit of the metal in the webs. (See record in Professor Burr's report.)

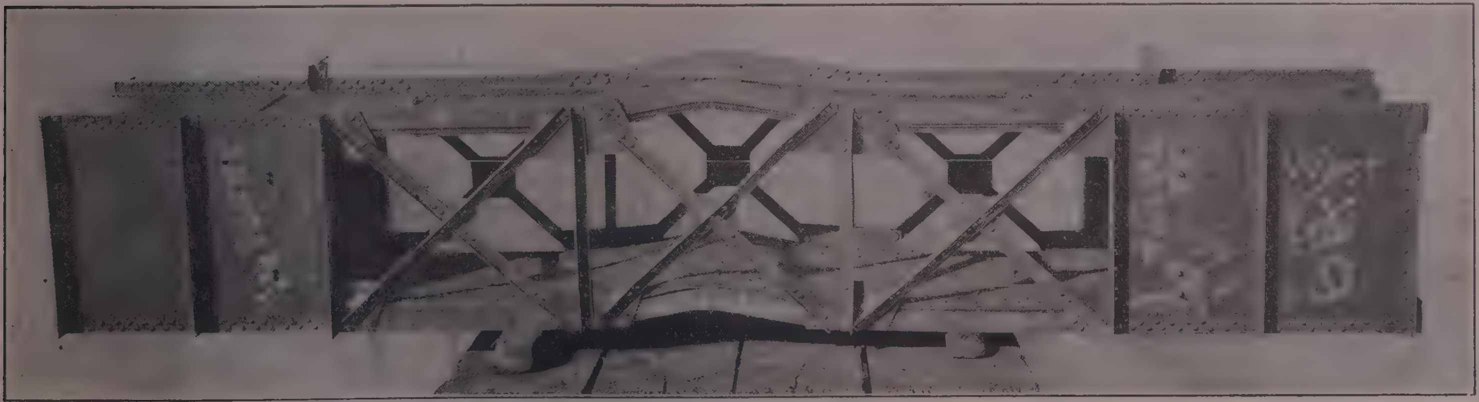
In this test the column seems to have been loaded evenly and centrally since the latticing was not seriously stressed.

The following notes concerning this test are of interest:

(1) There was some reason to think that chord A-9-L might have been bent sharply at the edge of the cover plate previous to failure. The inclination of its webs to the center line on Aug. 27 was very marked near the 8-9 joint (see drawing No. 28, Engineering Record, April 11, page 505). A series of straight edge measurements (see drawing No. 24) was made during the test of model chord No. 2 to determine whether any angle developed at the edge of the cover plate as the pressure increased, but no such movement was detected.

(2) During the test the yielding of the lattice rivets was observed by means of match marks upon the lattices and ribs. The results noted are given on drawing No. 24. They indicate that the pressure was centrally applied and that the lattice bars were not seriously stressed when these observations were taken. Towards the end of the test the lattice bars in the end panels were distinctly bowed upwards owing probably to the compression of the webs.

(3) Scaling at the pin bearings was observed as shown on drawing No. 24, but this was about the same at all four surfaces at the bearings and



Appearance of Model Chord 2 at the Conclusion of the Test.

did not indicate that there was any racking strain on the chord.

(4) The dishing of the webs during the test is shown on drawing No. 24.

(5) It will be noted from the measurements on drawing No. 24 that there were practically no horizontal or vertical movements of the chord webs with reference to the chord ends to which the reference wires were attached.

On Jan. 20, the Commission made three tests on full size lattice bars of the chord No. 9 design, the particulars of which are given on drawing No. 27. The tests were made in the laboratory of Messrs. Wm. Sellers & Co., Philadelphia, and the results obtained under the skillful handling of Mr. Backstrom may be accepted without question.

The purpose of these tests was to determine the strength of the lattice bars and the amount of yielding of the various parts of the length as the tension increased.

It will be noted that in each case failure took place at the center of the lattice bar, and the accompanying table is of interest.

It will be noted from the table that the riveting was sufficiently strong to develop the full strength of the cut lattice bar, and by reference to drawing No. 27 it will be seen that yielding took place simultaneously in the rivet connections and in the reduced section at the center of the bar.

Results of Lattice and Rivet Tests.

3 full size bars tested by Wm. Sellers & Co., on Jan. 20th, 1908. Ultimate load in lbs.	3 short sections of full sized bars tested by the Phœnix Bridge Co., on Jan. 21st, 1908. Ultimate load in lbs.	4 tests on 2-7/8" rivets in single shear made by the Phœnix Bridge Co., on Nov. 26th, 1907. Ultimate load in lbs.	2 tests on 2-7/8" rivets in single shear made by the Phœnix Bridge Co., on Jan. 14th, 1908. Ultimate load in lbs.	3 tests upon 2-7/8" rivets in double shear made by the Phœnix Bridge Co., Nov., 1907. Ultimate load in lbs.
60,100	61,100	63,000	62,500	121,000
59,800	62,000	63,000	63,100	122,000
59,500	60,700	63,800		123,800
		64,700		

As the first test chord failed by shearing of the rivet connections, and as no indications of failure at the center of the lattice bars was noticed, the result of the tests on the full size lattice bars was unexpected.

A series of tests on the lattice bars of test chord No. 1 was made by direction of the Commission on Jan. 23. The small testing machine belonging to the Phœnix Iron Co. was used, but as this was not well equipped for the work, the results are not wholly satisfactory.

The observations which are given on drawing No. 25 show that up to the moment of failure there was little yielding either of the rivets or of the reduced central portion, and that the failure took place in each case by rivet shear. The results of the specimen tests on the material for the angles are given in Professor Burr's report.

The accompanying table gives the results of tests upon the 7/24-in. rivets:

Tests on 7/24-in. Rivets.

At Phœnixville, Nov., 1907. Two rivets in double shear. Ultimate load in lbs.	At Phœnixville, Jan. 21, 1908. Two rivets in single shear. Ultimate load in lbs.
16,000	7,500
15,600	8,700
16,000	9,000

Under light loads, the slip was much the same as in tension tests and as the loading increased the web of the I-beam and not the riveting gave way.

These tests were made for the purpose of obtaining information to throw light upon the fracture of chord A-9-L, Quebec Bridge. The discussion of the failure of this chord will be found in appendix No. 16.

(Signed) Henry Holgate,
Chairman.
J. G. G. Kerry.
J. Galbraith.

APPENDIX 16, REPORT OF ROYAL COMMISSION ON QUEBEC BRIDGE

A Discussion of the Theories of Built-Up Compression Members.

This discussion will be confined to columns of which the cross-section is rectangular in outline and which are built up of two or more parallel webs with stiffening angles, connected by lattice bars, tie plates, diaphragms, etc. In such columns the parallel webs carry the load and the connections serve a subsidiary purpose. For convenience the webs, considered apart from their connections, will be termed the web system and the connections the lattice system. In many bridges the continuous cover plates of the top chord belong to both the web system and the lattice system, inasmuch as they both carry load and serve as connections for the side plates.

In the design of the cross-section the arrangement and dimensions of the web system are first considered. Column formulas based on experiment are used for this purpose. These formulas give the average unit stresses under which columns fail in terms of their length and radius of gyration. This radius is taken in the plane in which the column will probably fail by buckling or bending. A factor of safety is used in the design and a suitable arrangement of the cross-section of the web system adopted. The web system, in short, is designed from column formulas or the plotted results of experiments from which these formulas are deduced.

The design of the lattice system is quite a different matter. As a rule, it depends on the judgment of the engineer guided solely by experience. He finds little or nothing in scientific text-books or periodicals to assist his judgment. Some lattice formulas are in existence, but they are not generally known and their utility is more or less doubtful owing to the uncertainty of the data and assumptions on which they are founded. The unsatisfactory nature of the column formulas upon which the web system is designed is a matter of common knowledge among engineers, but the column formulas may be considered to represent exact science in comparison with the lattice formulas.

The lattice system performs two distinct functions. In the case in which each web of the web system carries its share of the load (that is to say, when there is no transfer of load in any part of the column from one web to another) the lattice system simply acts as a side support to the web system and by means of it a long web is divided up into a number of short columns. The stresses thrown into the latticing by this action cannot be computed. In this case the load on the column is parallel to the axis, but not necessarily coincident with it, and the curvature is assumed to be negligible. When, however, the load is inclined to the axis of the column, the lattice system has a different function. The angle of inclination may vary from point to point along the column owing to the curvature of the column. This curvature may be due to original bends or to the action of the load or to both combined. If the curvature is sufficiently small the variation of inclination due to it will be negligible. There remains, however, the original inclination of obliquity, which is due to the method of application of the loads at the ends of the column. If the eccentricity of application is the same at each end, and in the same plane with the axis of the column, there will be no obliquity other than that arising from the curvature of the webs, which may be negligible. If, however, the eccentricities at the opposite ends are different or in different directions, the obliquity may be of considerable amount. If the curvature of the column be negligible, the obliquity arising from the eccentricity will be the same at every point. This obliquity causes a transfer of load throughout the whole length of the column from one web to another. This transfer of load is accompanied by longitudinal shearing stresses in the lattice system. The obliquity also causes transverse shearing stresses at every cross-section of the column.

If the lattice system is considered to be sufficiently stiff the longitudinal shearing forces can be derived from the transverse shearing forces by the ordinary processes of statics as applied to elastic solids, and from them the lattice stresses and the lattice cross-sections may be computed.

If θ is the angle between the direction of the column axis and that of the load, S the transverse shear and P the load,

$$S = P \sin \theta$$

and since in practical cases θ is small, this may be written

$$S = P \theta$$

if θ be expressed in radians or as the ratio of the total eccentricity to the length of the column.

This formula holds true also if the curvature of the column is great enough to require consideration. In such a case θ varies along the column, and in computations the column should be divided by cross-sections so close together that the difference in θ at the two neighboring cross-sections may be disregarded.

So far the problem is comparatively easy—with the next step difficulties begin. The question now is: what value of the obliquity shall be chosen for design?

Since the obliquity depends upon inequality in the eccentricities at the ends, the maximum difference must be decided upon for design. It would seem reasonable to assume for this purpose equal eccentricities in opposite directions so that if e be the assumed eccentricity at one end the maximum value of the obliquity will be given by

$$\theta = \frac{2e}{l}$$

l being the length of the column. Against this view, however, it may be urged that the chances of the maximum value ever being reached are extremely small and that, therefore, some smaller value should be chosen.

Evidently the strength of this objection depends upon the value assumed for the eccentricity. The safe maximum value to be assumed for e depends upon the excellence of the design, both of the column and of the splices, on the accuracy of workmanship and on the care and precision employed in the erection.

It is impossible to estimate with accuracy the value of e under any set of conditions, but reasonable limits for its values will doubtless be learned from experience and study. With bad work, and more especially bad fitting and weak splices at the butt joints, the value of e may be much greater than it need be under other conditions of construction. In design, however, good workmanship and strong splices should be assumed. Theoretically the cross-sections of the latticing should be designed so that with the assumed eccentricity the latticing and the web systems will get their ultimate safe stresses simultaneously. This condition will be satisfied if the unit stress in the latticing has the same factor of safety as the maximum compressive stress in the web system corresponding to the eccentricity.

Let P be the safe load, A the area of the cross-section, $p = \frac{P}{A}$, d the greatest diameter of the cross-section in the plane of the latticing, r its radius of gyration parallel to d, q the unit stress at the most compressed edge, *e* the eccentricity of the load *P*, then

$$q = p\left(1 + \frac{ed}{2r^2}\right)$$

(An equation which is generally true only within the limit of elasticity.)
and consequently

$$e = \frac{2r^2}{d}\,\frac{q-p}{p}$$

$$\Theta = \frac{2e}{l} = \frac{2}{l}\,\frac{2r^2}{d}\,\frac{q-p}{p}$$

$$S = P\Theta = pA\,\frac{2}{l}\,\frac{2r^2}{d}\,\frac{q-p}{p}$$

$$= A\,\frac{2}{l}\,\frac{2r^2}{d}\,(q-p)$$

In design all the quantities in the above expressions for *e*, Θ and *S* are fixed without difficulty with the exception of *q*, the extreme unit stress in the web system. It is evident from the formula for *S* that it becomes zero when $q = p$.

Now the maximum value of *q* for which the formula

$$q = p\left(1 + \frac{ed}{2r^2}\right)$$

holds is, in general, the elastic limit. Consequently as *p* approaches the elastic limit, *S* approaches zero. Evidently, when *p* is equal to the elastic limit, the load must be central and without obliquity, and no part of it can be transferred from one web to another without inducing stresses in the second web in excess of the elastic limit.

The function of the latticing in such a case is simply to stiffen the webs and, as has already been said, the accompanying lattice stresses cannot be computed. The condition necessary for a theoretical computation of the stresses in the latticing is that the difference between *p* and *q* be reasonably large. Lattice formulas, of course, fix a value of *q* in a fashion, but only tests and experience can determine whether or not these give economical and safe results. Direct tests are difficult to apply and unless great care is exercised incorrect inferences may be drawn from them. A lattice column placed in a testing machine may fail in the web system, but this is no indication that the lattice system is strong enough for service in a similar column when in use as a bridge member. It may be that the obliquity of the load was too small to develop the lattice strength, and considerably less than the obliquity that would be found to exist on the bridge. With a greater obliquity the column might fail through the lattice system under a much smaller load. In other words, the failure of the webs is an indication that the full strength of the column has been nearly developed. The failure of the latticing may not be such an indication. The full strength of the column can be developed only by axial loading, and under such loading comparatively weak latticing may serve to develop this strength.

The full strength of the latticing can be developed only by oblique loading. The column strength in this case must be less than under axial loading, as otherwise failure would occur first in the web system.

The failure of lower chord A 9-L Quebec Bridge is an example of an insufficient lattice system. The webs bent and the lattices failed under a load only three-fourths of the specified maximum working load.

Determination of Area of a Lattice Bar Cross-Section.—The bar must be designed to take equal stresses in tension and compression. Let *P'* represent the lattice stress, *A'* the tension section, *A''* the compression section, *q'* the unit stress in tension, *q''* that in compression. The unit stress *q''* must be computed by a column formula.

Now $P' = kS$, where *k* is a coefficient which can be calculated from the known arrangement and dimensions of the lattice and web system. This calculation will be taken up later.

Thus

$$P' = A'q' = A''q'' = kS$$

$$\therefore A' = \frac{kS}{q'} = kA\,\frac{2}{l}\,\frac{2r^2}{d}\,\frac{q-p}{q'}$$

$$A'' = \frac{kS}{q''} = kA\,\frac{2}{l}\,\frac{2r^2}{d}\,\frac{q-p}{q''}$$

q' and *q''* in the design should have at least the same factor of safety as *q*.

It may be more convenient in many cases to make *A'* represent the shearing area of the rivets and *q'* the shearing unit stress. The net area of the lattice bar has been selected in this discussion because in the Quebec Bridge it was weaker than the rivet areas, the lattice bar section being 1.15 sq. in. and the rivet area 1.80 sq. in. (three rivets).

In the arrangement of the lattice system the free portions of the webs should have a value of *l/r* less than that of the column as a whole.

Lattice Formulas.—In the foregoing discussion it has been shown that there are two points with regard to which there must be more or less doubt and in which no aid can be expected from theory: first, the stresses to which the latticing is exposed when the load is axial, and, second, the value to be assumed for the maximum unit stress in the web system.

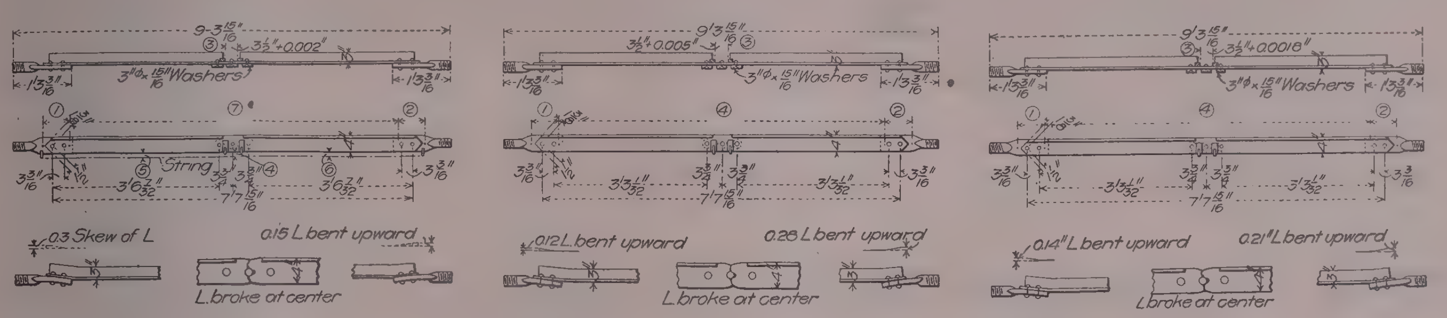
The assumption is made that a satisfactory solution of the second difficulty is sufficient to provide for the first. All practical lattice formulas determine, in effect, the value to be assigned to *q*. The same factor of safety is used for *q'* and *q''* as for *q*.

Mr. C. C. Schneider, consulting engineer, has called the attention of the Commission to an article on this subject by Professor Prandtl, of Goettingen, in the "Zeitschrift des Vereines Deutscher Ingenieure" of Dec. 28, 1907. Professor Prandtl assumes that the equation

$$q = p\left(1 + \frac{ed}{2r^2}\right)$$

holds up to the point of failure of the outer web. He makes *q* the ultimate strength of the portion of the outer web between neighboring lattice points. He necessarily makes *q'* and *q''* in the formula for lattice bars represent ultimate strengths. He also discusses the allowance to be made in the value of *r* on account of want of stiffness in the latticing. In other respects his discussion corresponds to ours.

In the same journal a letter appears from Professor Engesser, of Karlsruhe, who also refers to the diminution of *r* owing to want of stiffness of the latticing. This letter does not contain sufficient information to enable a reader to follow the line of thought. However, in the formula for



Meas'r'mts.	1	2	3	4	5	6	7
Orig. lgth. 11.15 a. m.	6.75	6.75	0.002	0.75			6' 11 11/16"
Load, total.							
a 5,000	6.76	6.76	0.002	—			
10,000	6.76	6.76	0.0047	0.75			
15,000	6.76	6.76	0.0071	—			
20,000	6.76	6.76	0.009	0.76			
22,500	—	—	0.0092	—			
25,000	6.76	6.77	0.0117	0.74			
27,500	6.77	6.78	——	—			
30,000	6.77	6.79	0.0119	0.73			
32,500	6.78	6.79	0.013	0.73			
35,000	6.78	6.79	0.0139	—			
37,500	6.78	6.80	0.0148	—			
Load off,							
12 noon	6.78	6.79	0.004	0.76			
No load,							
1:45 p.m.	6.78	6.78	0.004	0.76			
40,000	6.79	6.80	0.0166	0.73	0.79	0.67	
42,500	6.79	6.80	0.0182	0.74			6' 11¾"
45,000	6.81	6.80	0.0218	—			
47,500	6.82	6.80	0.0322	0.77			
50,000	6.85	6.80	0.0457	0.78			
52,500	6.86	6.84	0.0641	—			
55,000	6.92	6.86	0.0905	0.83	0.79	0.62	
57,500	6.97	6.90	0.1143	—			
60,000	—	—	0.191 (ab't)				
b 60,100	6.99	6.91	——	—			7' 0⅝"

a Total on. After reducing load to 100 lb., measurements were taken.
b Measured after breaking under load of 60,100 lb. at 2:35 P. M.

Measurements.	1.	2.	3.	4.
Orig. Length 3:10 P. M.	6.75	6.75	0.005	—
Load, total.				
a 5,000	6.75	6.75	0.010	6' 11⅜"
20,000	6.75	6.75	0.0159	—
25,000	6.75	6.75	0.018	—
30,000	6.75	6.75	0.0189	—
35,000	6.75	6.78	0.0202	—
37,500	6.76	6.80	0.0210	—
40,000	6.76	6.82	0.0218	6' 11 7/16"
42,500	6.76	6.84	0.0222	—
45,000	6.77	6.83	0.0231	b
47,500	6.77	6.83	0.0352	—
50,000	6.79	6.85	0.0527	6' 11 9/16"
52,500	6.81	6.87	0.0572	6' 11⅝"
55,000	6.83	6.91	0.103	6' 11 11/16"
57,500	6.86	6.97	0.1219	6' 11¾"+
c 59,800	6.87	6.96	—	d

a Total on. After reducing load to 100 lb., measurements were taken.
b At 44,500 lb. felt yielding at scale.
c Failed at 59,800 lb., 4:20 P. M.
d Measurement 4 before breaking 6 ft. 11 13/16 in., after breaking 7 ft. 0⅛ in.

Measurements.	1	2	3	4
Total Load				ft. ins.
a 5,000	6.74	6.74	0.0018	6 11⅝
25,000	6.74	6.74	0.0089	6 11 11/16
30,000	6.76	6.76	0.0101	6 11 11/16
35,000	6.76	6.76	0.0119	6 11⅞
37,500	—	—	0.0094	b6 11¾
40,000	6.77	6.78	0.0093	6 11¾
42,500	6.77	6.77	0.0102	6 11¾
45,000	6.78	6.79	0.0131	6 11¾
47,500	6.81	6.80	0.0190	6 11 13/16
50,000	6.82	6.83	0.0355	6 11 13/16
c 50,000	6.83	6.85	0.040	6 11⅞
52,500	6.86	6.88	0.0611	6 11⅞
55,000	6.89	6.90	0.075	6 11 15/16
57,500	6.94	6.92	0.103	7 0+
d 59,500	—	—	——	7 0 7/16

a Total on. After reducing load to 200 lb., measurements were taken.
b Changed center support.
c Measurements after holding load for 10 min. longer.
d Failed at 59,500, 5:45 P. M.

Drawing 27. Report of Royal Commission on Quebec Bridge Failure.

Results of Tests of 3 x 4 x 3-8-In. Angles Made for the Commission at Wm. Sellers & Co., Philadelphia.

lattice bar areas, he seems to replace $\frac{q-p}{q'}$ and $\frac{q-p}{q''}$ by $\frac{q-p}{q}$ in which q is the ultimate strength of the material and p is the ultimate strength of the column which he determines by use of the Tetmajer formula. He also replaces the factor 2 by π.

He states that he published his investigations in 1891 and 1893.

Mr. H. S. Prichard, in The Engineering Record, Oct. 12, 1907, gives a rule which he had used for several years, which makes

$$S = .015\,P$$

$$\text{in other words } \Theta = .015$$

Mr. Szlapka, in his evidence, states that after a most painstaking search the only information he could find on the subject of lattice computations was that given in Johnson's "Modern Framed Structures."

The experience of the Commission is practically the same as that of Mr. Szlapka for, except the rule in "Modern Framed Structures," all the information we have been able to find has appeared in the periodical press since the collapse of the Quebec Bridge.

Mr. Bindon Stoney, one of the earlier authorities on bridge construction, has given a method of computing statically the stresses in lattices existing in a bent column.

The article in "Modern Framed Structures" is as follows:

"There are no rules other than empirical ones in use by which the size and spacing of lattice bars for compression members are determined. . . . . It has been suggested that, as our compression formulas all assume a certain extreme fiber stress due to the flexure of the strut, from this known extreme fiber stress we find an equivalent uniform load acting in the plane of the latticing which will produce this fiber stress and from this load find the stress in the lattice bars."

This method is equivalent to assuming that $q = b$ and $\Theta = \frac{4e}{l}$, f being taken from the formula $p = f - c\,\frac{l}{r}$ or the formula $p = \frac{f}{1 + \frac{1}{c'}\left(\frac{l}{r}\right)^2}$ when applied to the working stresses.

The value of S thus becomes

$$S = P\,\Theta = P\,\frac{4e}{l} = P\,\frac{8r^2}{ld}\,\frac{f-p}{p}$$

From the straight line formula

$$\frac{f-p}{p} = \frac{c}{p}\,\frac{l}{r} = \frac{A}{P}\,\frac{cl}{r}$$

So that

$$S = \frac{P\,8r^2}{ld} \cdot \frac{Acl}{Pr} = \frac{8Acr}{d}$$

From the Rankine formula

$$\frac{f-p}{p} = \frac{1}{c'}\left(\frac{l}{r}\right)^2$$

$$\therefore e = \frac{2r^2}{d}\,\frac{1}{c'}\left(\frac{l}{r}\right)^2 = \frac{2l^2}{c'd}$$

$$\Theta = \frac{4e}{l} = \frac{8l}{c'd}$$

$$S = P\,\Theta = \frac{8l}{c'd}\,P$$

These equations give very different values for S, even though the constants of the Rankine formula be computed so that the curve represented by it and the straight line represented by the straight-line formula are tangent at the point corresponding to the value of l/r in question.

Mr. Szlapka used the rule in "Modern Framed Structures." He selected Rankine's formula and gave c' its value for square bearings. He, however, modified the method by using a central load instead of a distributed load. This modification had the effect of making the area of the lattice bar one-half of that given by the method suggested in "Modern Framed Structures."

Mr. Szlapka finally adopted a larger cross-section than his method gave, and one which, in his judgment, was sufficient.

If he had tested the method fully he would have found it capable of giving areas ranging up to ten times the area computed by him, a result which would have shown the unreliability of this method. He might, of course, have come to the conclusion that a rule capable of giving such different results was valueless.

In an article in "Engineering," Sept. 27, 1907, Professor Keelhoff, of Ghent University, states that in 1893 he developed a lattice formula which has been more or less extensively used. When thrown into the form of the theoretical formula previously given, the following results are obtained:

Professor Keelhoff multiplies the expression for Θ by the coefficient $\frac{\pi}{2}$. Thus, instead of $\Theta = \frac{2e}{l}$ his method gives $\Theta = \frac{\pi e}{l}$, this change being the result of a theoretical study in which he adopted Euler's sinusoid curve as the probable form of a bent column. He also replaces q by f taken from the column formula for working stresses

$$p = f - c\,\frac{l}{r}.$$

It is thus apparent that the methods of lattice computations which have been used in practice, when thrown into the form adopted in this discussion, simply assign values to the unknown q, and in some cases multiply the theoretical obliquity by the factor 2 or $\pi/2$.

The following is a résumé of the lattice formulas discussed in this appendix:

THEORETICAL FORMULAS.

$$e = \frac{2r^2}{d}\,\frac{q-p}{q}$$

$$\Theta = \frac{2e}{l} = \frac{2}{l}\,\frac{2r^2}{d}\,\frac{q-p}{p}$$

$$S = P\,\Theta = p\,A\,\frac{2}{l}\,\frac{2r^2}{d}\,\frac{q-p}{p}$$

$$= A\,\frac{2}{l}\,\frac{2r^2}{d}\,(q-p)$$

$$A = \frac{kS}{q'} = k\,A\,\frac{2}{l}\,\frac{2r^2}{d}\,\frac{q-p}{q'}$$

$$A'' = \frac{kS}{q''} = k\,A\,\frac{2}{l}\,\frac{2r^2}{d}\,\frac{q-p}{q''}$$

(q to be determined by judgment and not to exceed the elastic limit.
q' and q'' to have the same factor of safety as q.)

FORMULAS USED IN PRACTICE.

Prandtl makes q the ultimate strength of the portion of the outer web between neighboring lattice points and q' and q'' the ultimate strength of the lattice bars used.

Engesser replaces $\frac{q-p}{q'}$ and $\frac{q-p}{q''}$ by $\frac{q-p}{q}$ respectively, using ultimate values. He also multiplies Θ by $\frac{\pi}{2}$ i. e., makes $\Theta = \frac{\pi e}{l}$

Prichard makes Θ constant $= .015$.

"Modern Framed Structures" makes $q = f$ of the working stress formula $p = f - c\,\frac{l}{r}$ or

$$p = \frac{f}{1 + \frac{1}{c}\left(\frac{l}{r}\right)^2}$$

And also multiplies Θ by 2, i. e., makes $\Theta = \frac{4e}{l}$

Szlapka modified the rule in "Modern Framed Structures" by not using the multiplier 2 in the value of Θ, i. e., made $\Theta = \frac{2e}{l}$ and also used the formula

$$p = \frac{f}{1 + \frac{1}{c'}\left(\frac{l}{r}\right)^2}$$

giving c' its largest value, viz., 36,000.

Keelhoff makes $q = f$ of the working stress formula $p = f - c\,\frac{l}{r}$ and also multiplies Θ by $\pi/2$.

Computation of k in formula $P' = k\,S$.—The method of making this computation will be illustrated by a numerical example.

For this purpose lower chord A 9-L Quebec Bridge will be taken. On the assumption that the latticing is sufficiently stiff to enable the webs to act as a unit, the relation between the longitudinal shear S' in the length of one panel of latticing and the transverse shear S at the end of the panel is given by the statical formula $S' = \frac{SxQ}{I}$ where x is the length of one lattice panel, Q the moment of area about the central axis of the chord cross-section perpendicular to the lattice planes of that portion of the web cross-section which lies outside the given plane of longitudinal shear, and I the moment of inertia of the whole chord cross-section about the same axis.

Evidently maximum S' corresponds to maximum Q, which in chord A 9-L, occurs between the center webs. The numerical values are $Q = 6439$, $x = 72.75$, $I = 302640$, dimensions being given in inches. Therefore, $S' = 1.55\,S$ between the center webs. Similarly between the outside web and the center web $Q = 5313$, giving

$$S' = 1.28\,S.$$

In a lattice panel these are four bars arranged two and two as the diagonals of a square of which the side is 54.36 in., this being the distance between the axes of the outside webs.

Therefore $P' = \frac{S'}{4} \times \sqrt{2} = .35\,S'$

$P' = .35 \times 1.55\,S = .54\,S$ between the center webs,

and $P' = .35 \times 1.28\,S = .45\,S$ between an outside web and a center web.

Thus the values of k are .54 and .45.

From the design of the chord it is evident that the net area of the lattice bar is governed by $k = .54$, while the rivets connecting the bar to the outside web are determined by $k = .45$ and those connecting the bar with the inside web by the difference of these values, which is .09. That is to say, if five rivets were necessary to connect the bar to the outer web, only one would be required for the connection with the inner web.

Transverse shears and bending moment exist in the webs due to the transverse shear S on the cross-section of the chord.

The maximum transverse shear in the outer web occurs in the space between the quadrilateral panels of latticing. It is equal to $\frac{1}{2} \times \frac{54.36}{72.75} \times 1.28\,S = \frac{1}{2} \times .747 \times 1.28\,S = .48\,S$, if the small bending moments in the webs due to the assumed distribution of stress at the plane of section be neglected; the maximum shearing stress on a center web section is thus .02 S, the sum being .50 S, half the shearing stress on the cross-section of the chord.

Thus 96 per cent of the transverse shear is carried by the outer webs and only 4 per cent by the inner webs in the space between the panels.

The difficulty of determining theoretically the values to be assigned to the quantities $\frac{q-p}{q'}$ and $\frac{q-p}{q''}$ in the formulas

$$A' = k\,A\,\frac{2}{l}\,\frac{2r^2}{d}\,\frac{q-p}{q'}$$

$$\text{and } A'' = k\,A\,\frac{2}{l}\,\frac{2r^2}{d}\,\frac{q-p}{q''}$$

has been pointed out. It will be of interest to compare the solutions of this problem as given by the various methods that have been described in a numerical example, and also to compare the corresponding lattice cross-sections.

For this purpose the web system of lower chord A 9-L, Quebec Bridge, will be selected. It will be sufficient for the present purpose to consider the formula for the tension section A'.

In this chord, $l = 684$ in., $r = 19.7$ in., $d = 67.5$ in., $A = 780$ sq. in., $k = .54$.
Thus

$$A' = .54 \times 780 \times \frac{2}{684} \times \frac{2 \times 19.7^2}{67.5} \times \frac{q-p}{q'}$$

$$= 14 \frac{q-p}{q'} \text{ sq. in.}$$

Prandtl:

$\frac{l}{r}$ for outer web between lattice points $= 44$; ultimate strength of outer web, say $(48{,}000 - 210 \times 44) = 38{,}760$.

Specified unit load on column $p = 24{,}000$

Tensile strength of lattice bar $q' = 60{,}000$

$$\therefore \frac{q-p}{q'} = \frac{38760 - 24000}{60000} = \frac{14760}{60000} = .25$$

$$A' = 14 \times .25 = 3.50 \text{ sq. in.}$$

If the unit load for the column had been determined by the formula $p = (16{,}000 - 70 \frac{l}{r}) = (16{,}000 - 70 \times 34.7) = 13{,}571$, we should have had

$$\frac{q-p}{q'} = \frac{38760 - 13571}{60000} = \frac{25189}{60000} = .42$$

and $A' = 14 \times .42 = 5.88$ sq. in.

Engesser:

$$q = q' = 60{,}000$$

$$p = 48{,}000 - 210 \frac{l}{r} = 48{,}000 - 210 \times 34.7 = 40{,}713$$

$$\frac{q-p}{q'} = \frac{60000 - 40713}{60000} = \frac{19287}{60000} = .32$$

$$A' = \frac{\pi}{2} \times 14 \times .32 = 7.04 \text{ sq. in.}$$

When this formula is used, the cross-section of the latticing does not vary with the load.

Prichard:

$$\theta = .015$$

Now theoretically $\theta = \frac{2}{l} \frac{2r^2}{d} \frac{q-p}{p}$

$$= \frac{2}{684} \times \frac{2 \times 19.7^2}{67.5} \times \frac{q-p}{p}$$

$$= .0333 \frac{q-p}{p}$$

$$\therefore \frac{q-p}{p} = \frac{.0150}{.0333} = .45$$

$$\therefore \frac{q-p}{q'} = \frac{q-p}{p} \frac{p}{q'} = .45 \frac{p}{q'} = \frac{.45 \times 24000}{40000} = .27$$

Assuming that 40,000 in tension represents the same factor of safety as 24,000 (in the case of this column) in compression

$$A' = 14 \times \frac{q-p}{q'} = 14 \times .27$$

$$= 3.78 \text{ sq. in.}$$

Since $\frac{p}{q'}$ is constant for all factors of safety this result applies to all loads.

"Modern Framed Structures":

(1) Straight-line formula:

$q = 24{,}000$ $\quad p = 24{,}000 - 105 \times 34.7 = 20{,}356$

$q' = 30{,}000$

$$\therefore \frac{q-p}{q'} = \frac{24000 - 20356}{30000} = \frac{3644}{30000} = .12$$

$$A' = 2 \times 14 \times \frac{q-p}{q'} = 28 \times .12 = 3.36 \text{ sq. in.}$$

In this formula evidently $\frac{q-p}{q'}$ is constant for all factors of safety, and, therefore, for all loads.

(2) Rankine's formula:

$$\frac{q-p}{q'} = \frac{f-p}{q'} = \frac{\frac{p}{c'}\left(\frac{l}{r}\right)^2}{q'}$$

assuming $p = 24{,}000$, $c' = 18{,}000$, $\frac{l}{r} = 34.7$, $q' = 30000$.

$$\frac{q-p}{q'} = .0535$$

$$A' = 2 \times 14 \times \frac{q-p}{q'} = 28 \times .0535 = 1.50 \text{ sq. in.}$$

If c' be made 36,000 the values are

$$\frac{q-p}{q'} = .0267 \qquad A' = .75 \text{ sq. in.}$$

Evidently the same results will be given by all factors of safety, i. e., for all loads.

Mr. Szlapka's method, if he had used the proper value of k, would have given $A' = .37$ sq. in. He assumed that the panels of latticing were square whereas they were oblong and the lattice bars were not diagonal.

In the actual design, however, he made $A' = 1.15$ sq. in.

Keelhoff:

$$\frac{q-p}{q'} = \frac{f-p}{q'} = \frac{24000 - 105 \times 34.7}{30000} = \frac{24000 - 20356}{30000} = \frac{3644}{30000} = .12$$

$$A' = \frac{\pi}{2} \times 14 \times \frac{q-p}{q'}$$

$$= 1.57 \times 14 \times .12 = 2.64 \text{ sq. in.}$$

The value of $\frac{q-p}{q'}$ will not be altered by using different factors of safety for p, q and q' and therefore applies to all loads. These results, arranged for comparison, are collected in the following table:

Author.	$\frac{q-p}{q'}$	A' sq. in.
Prandtl	.25	3.50 for p = 24,000.
	.42	5.88 for p = 13,571 from formula $p = 16{,}000 - 70 \frac{l}{r}$
Engesser	.32	7.04 for all values of p.
Prichard	.27	3.78 for all values of p.
"Modern Framed Structures" ..	.12	3.36 for all values of p, straight formula.
"Modern Framed Structures" ..	.0535	1.50 for all values of p, Rankine's formula, c' = 18,000.
"Modern Framed Structures"	.0267	.75 for all values of p, Rankine's formula, c' = 36,000.
Keelhoff	.12	2.64 for all values of p.

Mr. Szlapka, in his design, made A' = 1.15 sq. in.

The following list shows the value of $\frac{q-p}{p'}$, in A 9-L adjusted by multiplying the original values by the factors $\pi/2$ and 2, where necessary, for use in the formula

$$A' = k A \frac{2}{l} \frac{2r^2}{d} \frac{q-p}{q'}$$

Prandtl:	$\frac{q-p}{q'}$	= .25	p = 24,000.
"	"	= .42	p = 13,571. from $p = 16{,}000 - 70 \frac{l}{r}$
Engesser:	"	= .50	for all values of p.
Prichard:	"	= .27	for all values of p.
"Modern Framed Structures"	"	= .24	for all values of p, straight line formula.
"Modern Framed Structures"	"	= .1070	for all values of p, Rankine's formula.
"Modern Framed Structures"	"	= .0535	for all values of p, Rankine's formula.
Keelhoff:	"	= .19	for all values of p.

The practical formulas thus give values for the net section of a lattice bar in chord A 9-L ranging from .75 sq. in. to 7.04 sq. in.

The rule in "Modern Framed Structures" is capable of giving values ranging from .75 sq. in. up to 3.36 sq. in.

The range of values is even more indefinite than the numerical values indicate, depending as it does on the varying opinions regarding the values to be assigned to the constants of the column formulas.

It is evident that the number of rivets necessary to develop the values of the larger sections given above would make the use of lattice bars impossible. Cover plates and horizontal diaphragms would be required.

The value of l/r for the outer web in chord A 9-L is 44 and for the column as a whole, 34.7. This is not good design as the first value ought to be less than the second.

The latticing between the center webs is inefficient. Intermediate latticing should have been used on these webs. One of the bars between the center webs in every panel on the upper face of the chord A 9-L has a net section of only 1.15 sq. in. at the center and 1.5 sq. in. for a length of 4½ in., whereas between the center web and the outside web the section is 2.48 sq. in.

The bending moments and shears in the webs, the bending moments in the latticing and the compressive stresses in the latticing due to the load on the column have not been considered in this discussion. The theory of the design of latticing has been discussed on the assumption that the curvature in the column under load is negligible, as it ought to be.

When appreciable bending occurs, the total transverse shear is still given by the formula $S = P\theta$. On the other hand, when the curvature of the axis of the column varies from point to point, the longitudinal shear S' will not be as great in comparison with S as if the column remained straight, on account of part of the transverse shear being balanced by the resistance to bending of the webs taken individually. Only the difference between these actions is thrown into the latticing and represented by S'.

A method of dealing with the shears which in some respects is simpler than that adopted might have been used. This simpler method is based on the assumption that the small bending stresses at the ends of the webs in a panel of latticing, due to the assumed unity of the column, may be neglected. In this case q will denote the average unit stress in the outer web under eccentric loading and not the extreme unit stress in this web. This method has, indeed, been partially applied in this appendix. The results do not differ appreciably from those of the general method adopted.

Failure of lower chord A 9-L.—In discussing this failure the original conditions will be assumed to hold; that is, $S' = .54S$ and $.45S$ between the inner webs and between the inner and outer webs, respectively. It is possible that these two values were closer together, owing to the working loose of the latticing between the inner webs.

Assuming $P =$ the load at the time of failure $= 14{,}000{,}000$ lb., and $P' = 50{,}000$ lb., a load sufficient, according to experiments made at Philadelphia, to cause slow movement and rivet slip,

$$\text{then } S = \frac{50{,}000}{.54} = 92{,}592$$

$$\text{Now } S = P\theta$$

$$\therefore \theta = \frac{S}{P} = \frac{92{,}592}{14{,}000{,}000} = .0066$$

Thus, if the obliquity $= .0066$ existing under a load $P = 14{,}000{,}000$ lb., the chord would gradually go to destruction.

The measurements made by Messrs. Birks, McLure and Kinloch on Aug. 27, 1907, show when averaged up for the four webs, a deflection of the chord as a whole of 1¾ in. at the point between the second and third lattice panels from the south end. Since no measurements were made to determine the position of the axis of the chord from panel point to panel point, it is impossible to state the real deflection of the chord, and the only assumption which can be made is that it is represented by the above amount.

It is not possible to state why the maximum deflection took place at the point mentioned. There may have been an original deflection of small amount there, a defect in workmanship or a local injury from the fall mentioned in appendix No. 11. [See The Engineering Record, March 14, 1908.]

The accompanying buckling of chords 8 R and 9 R, cantilever arm, shows that the failure itself was not accidental, although it may have been localized by defects or accident. The Philadelphia tests show that slip in the lattice system would have commenced with an obliquity less than one-half of that obtained above and it is possible that the first movement was caused by a combined stress due to a small initial obliquity and to the shortening of the chord under stress.

It is hardly necessary to note that, owing to the form taken by the chords, partial failure in the lattice system must have preceded any failure in the web system.

If now it be assumed that the chord had an original deflection of ½ in. at the place in question, the inclination of the axis of the chord in the first panel of latticing will be found to be $5/191 = .0026$, which is greater than in any other panel. To make up the required obliquity of .0066 it will, therefore, be necessary to assume that the direction of the load originally had an obliquity of $.0066 - .0026 = .0040$ to the axis. This would be equivalent to 2¾ in. in the length of the chord, probably due to an eccentricity of about ½ in. to the east at panel point 8-9 and 2¼ in. to the west at panel point 9-10.

From the discussion given in this appendix and from the results obtained in the test of model chord No. 1, it is not difficult to see that failure was certain and close at hand on Aug. 27. The evidence shows that the increase of obliquity which created this dangerous condition took place between Aug. 24 and Aug. 27.

On Aug. 27 the curvature was such that the line of load which would give the least maximum obliquities in the chord had an eccentricity with regard to the center line adopted for the measurements toward the west of 1¾ in. at panel point 8-9 and of ¾ in. to the east at panel point 9-10. As this line of load gives minimum lattice stresses, we assume it, for the purpose of this investigation, to be the true line of load. The inclination of the axis of the chord in the first panel of latticing at the south end with reference to the same center line was about .016, thus making the obliquity of the line of load in this panel about $.016 - .004 = .012$.

The question now occurs: how was it possible that the chord could sustain an obliquity of .012 when an obliquity of .0066 was sufficient to strain the lattices to the danger point?

If the chord had remained straight, an obliquity of $.0066 \times \frac{60000}{50000} = .0079$ would have caused immediate failure.

In reply it may be said, as has already been pointed out, that the consideration of the bending moments in the individual webs accompanying the bending of the chord, the bending moments in the latticing, the compressive stresses in the latticing due to the load on the columns, etc., etc., has heretofore been omitted. Of these the first appears to be the most important and its effect in aiding the lattice system may be estimated as follows:

Let M denote the increase in the bending moments of the outer web in the first panel of latticing at the south end of the chord, M' the corresponding quantity for the inner web,

then $$54.36\, S' = 69\, S - 2\,(M + M')$$

$$\therefore S' = \frac{69\, S - 2\,(M + M')}{54.36}$$

the length of this panel being 69 in.

Now the chord in the length of the first two panels of latticing, viz., in 142 in., has a central deflection of ¼ in. The radius at the middle point may thus be computed approximately. The resulting value is $r = 10{,}000$ in. The true radius may be less than this as the web was probably nearly straight next the cover plate with increasing curvature towards the point of greatest deflection. It is even possible that there was a point of contraflexure near the edge of the cover plate.

Now

$$M = \frac{EI}{r} = \frac{30{,}000{,}000 \times 366}{10{,}000} = 1{,}098{,}000 \text{ in.-lb.}$$

$$M' = \frac{EI'}{r} = \frac{30{,}000{,}000 \times 239}{10{,}000} = 717{,}000 \text{ in.-lb.}$$

From what has been said with reference to the change of curvature along the length of the chord, it is not unreasonable to assume that the above bending moments represent approximately the increase of bending moment in the length of the first panel of latticing, i. e., in 69 in. from south to north.

Thus

$$2\,(M + M') = 2\,(1{,}098.000 + 717{,}000) = 3{,}630.000$$

$$\therefore S' = \frac{69\, S - 3{,}630{,}000}{54.36}$$

Now

$$S = P\,\theta = 14{,}000.000 \times .012 = 168{,}000$$

$$\therefore S' = \frac{69 \times 168{,}000 - 3{,}630{,}000}{54.36} = 146{,}468$$

And

$$P' = \frac{1.4}{4}\, S' = .35 \times 146{,}468 = 51{,}264 \text{ lb.}$$

Thus on account of the resistance to bending of the individual webs, the obliquity .012 will produce a stress in the lattice bars of about 51,264 lb., whereas had the webs remained straight, an obliquity of only .0079 would have destroyed the lattice bars.

In the tension experiments described in appendix No. 15 on lattice bars like those used in the Quebec Bridge, the breaking values of P' were 60,100, 59,800 and 59,500 lb.

In the above calculations the compressive stresses in the lattice bars due to the compression of the chord as a whole have been neglected.

The above explanation of the failure of this chord under three-quarters of its maximum working load contains assumptions which render it only tentative. It indicates the dangerous effects of even small obliquities and deflections on the safety of a chord with weak latticing. It is quite probable that the obliquity was in great measure due to movements at the field joint in panel 9 L, which was riveted up, and at the field joint in panel 10 L, which was being riveted up at the time of the collapse. In fact, all the troubles in the lower chords of both anchor and cantilever arms which developed after Aug. 6, 1907, seem to be partly attributable to movement at the field joints. These movements were noticed principally in the inner webs, which have much less horizontal stiffness than the outer webs. These webs were intended to carry the same unit loads as the outer webs, and yet at the field joints they were connected to the cover plates with only half as many rivets, the small web angles used not permitting more. The outer webs with heavy angles and fairly effective latticing seem to have stood up under the stresses—the small angles and inefficient splicing and latticing of the inner webs allowing them to yield, thus disturbing the intended action at the field joints and panel points and giving opportunities for unforseen eccentricities of loading. Heavier angles on the center webs under the cover plates, heavier splicing and heavier top and bottom cover plates would have added much to the efficiency of the joints.

An important function of cover plates is that they maintain the webs or ribs at their proper distances apart, but in erection, the bottom cover plate was taken off during the riveting up of the joint, and was replaced by small angle bars which were entirely too slight to perform the function of the cover plate. This is shown by the fact that a much greater movement was noticed at the bottom of the center web than at the top.

See drawings Nos. 25 and 26 [omitted], 27, 28, 29 and 30. [See The Engineering Record, April 11, 1908.]

Test of Model Chord No. 1.—Made at Phoenixville by the Phoenix Bridge Co., Nov. 21, 1907.

This chord was esentially a model of chord A 9-L, Quebec Bridge, between panel points. It had, however, no field joint. Its dimensions were one-third of those of chord A 9-L. (See drawing No. 22.) It broke without warning under a load $P = 2{,}322{,}600$, by the failure of the outside lattice rivets.

The ultimate shearing value of one rivet was 4,000 lb. The lattice angle was connected with the web by two rivets.

From the foregoing formulas, therefore,

$$S = \frac{P'}{k} = \frac{8000}{.45} = 17{,}778 \text{ lb.}$$

$$\theta = \frac{S}{P} = \frac{17{,}778}{2{,}322{,}600} = .0077$$

The obliquity of the load which caused the failure was .0077, subject to correction for error of calibration of the testing machine.

Test of Model Chord No. 2.—Made at Phoenixville by the Royal Commission, Jan. 18, 1908.

The test of chord No. 1 showed that the lattice system was too light, but gave no indication of the ultimate strength of the column if properly latticed. The capacity of the Phoenix Iron Co.'s machine was not sufficient to permit a complete test of this kind. In order, therefore, to get results, a chord with only two webs was constructed. The dimensions of the webs were one-third of those of the outer webs of chord A 9-L. (See drawing No. 23.)

The lattice system, however, was made about twice the strength of that in model chord No. 1 and the length of the model was only 11 ft. 4¼ in. to c. of pinholes. The lattice bars were connected to the web by four rivets instead of two.

This chord fulfilled the expectations of the Commission and broke under a load of 37,000 lb. per square inch by the yielding of the webs in the center panel.

From what has been said it is evident that this experiment did not settle the question of the strength of the latticing. Stronger latticing might have been required in good design. The proper inference is that the obliquity was too small to break the latticing, so that the full strength of the webs was nearly, if not quite, developed.

Since the inside webs of the Quebec chords are less stiff than the outer webs, it seems to be a fair inference that 37,000 lb. per square inch is higher than the strength of the Quebec chord would have been, even if properly latticed.

Some allowance also must be made for the higher strength and elastic limit of the small plates and angles used in these models as compared with those in the bridge.

There is doubt as to the correctness of the calibration of the testing machine, so that the above figures are subject to correction. In the tests of both models, the dishing of the webs between the upper and lower lattice systems was small and only careful measurements rendered its existence apparent. (See drawings Nos. 21, 22, 23 and 24.)

In concluding this appendix, some brief comment is necessary upon two points: (1) The use made by Mr. Szlapka of the information existing in 1903 respecting the design of latticing and (2) the application in practice of the theoretical formula given in our discussion.

(1) The use made by Mr. Szlapka of the information existing in 1903 respecting the design of latticing. It has been admitted by Mr. Cooper that he failed to give the design of the lower chords the degree of personal attention that he gave to the details of the tension system. The foregoing discussion shows that even at the present time theories of lattice design are seriously in conflict, and the strength of any lattice system will vary materially according to the formula adopted. Mr. Szlapka used, with his own modifications, the only system of lattice computation generally known to American engineers. This method involved the choice of a column formula from which

to determine certain quantities necessary in the lattice computations. Mr. Szlapka selected the column formula adopted by his own company and used the constants for it that, in his judgment, were most in keeping with the conditions of the case and in best accord with the spirit of the specification. He made what he considered a liberal increase in his adopted sections over what his computations called for. The result has shown that his judgment was faulty, but we are not prepared at this date to define the maximum safe sections for the latticing for these chords. The profession has learned much from Mr. Szlapka's mistake, but it is not even yet in a position to determine the percentage of his error. The lattices of model chord No. 2 were proportionately only 50 per cent. heavier than those used on the Quebec chords, and yet they did not fail until the webs buckled. We have indicated in the discussion that Mr. Szlapka's attention would soon have been drawn to the weakness of the theory by which he was guided, had he made any study of the results given by that theory with different assumptions. No explanation, except the previous uniform success of compression members in service, can be offered for his failure to do this.

(2) The application in practice of the theoretical formula given in our discussion depends upon our ability to select values of q and p suitable to the detail of construction in the special column under consideration. The values of p are determined in practice by the use of column formulas, but no one contends that the range of the tests upon which these formulas are based is sufficiently extensive to cover all the conditions that affect column strength; the formulas are simply accepted as the best guide that we now have. The maximum value of q will be fixed by the characteristics of the metal to be used. It is evident that by experience values of q and p may be gradually determined, which will make it possible to design latticing that will be unquestionably safe and not unnecessarily heavy. We may here point out that great compression members, such as the Quebec Bridge chords, call for just as much individual study in design as an ordinary small bridge, and that any specification for such members should give reasonable latitude for the exercise of judgment by the designing engineer.

An Argentine Railroad, 176 miles long, was recently opened between Jujuy and La Quiaca, on the Bolivian border. The highest elevation reached is 12,100 ft. For 6 miles cog-wheel locomotives are used.

A High Pressure Water Works.

One of the highest pressure water works pumping stations in the United States is situated in Morenci, Graham County, Arizona, and is owned by the Morenci Water Co., of which Mr. C. P. Chester is superintendent. The water, which is of excellent quality, is obtained from Eagle Creek, a small stream flowing through a deep canyon five miles from town. The reservoir supplying the town is 1,717 ft. above the pumping station. They are connected by a 6-in. standard steel pipe pressure main, laid on top the ground over very rough country. The static head combined with the friction through the pipe requires a pressure of from 800 to 885 lb. at the pumps. About 14 million gallons per month are pumped through this line. In laying the pipe the ordinary sleeve connections were used, except extra heavy faced flanges were used where bends occurred. No expansion joints were found to be necessary, as the many bends in the pipe allow it to move freely.

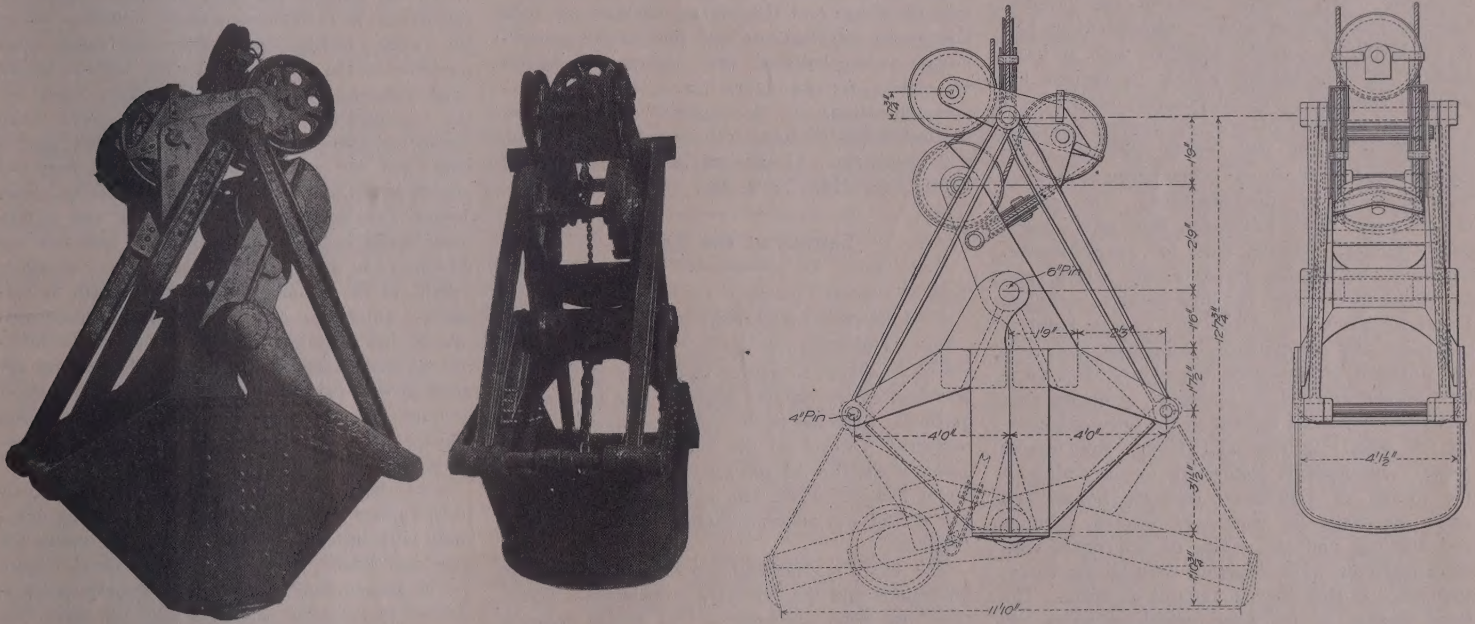
The power plant is of ordinary design, with Stirling boilers and a cross-compound condensing engine belt-driving four plunger-type power pumps. Texas oil is burned and it is found that about 13 lb. of oil will deliver 1,000 gal. into the reservoir. The method of piping the oil to the pumping station is perhaps unique. The oil is delivered from tank cars in Morenci directly into a pressure tank. A 4-in. oil line connects this tank with the oil reservoir at the pumping station. This oil line is laid over a hill, the highest point being level with the water reservoir supplying the town. Water from the town mains is turned into the bottom of the oil pressure tank and the oil is forced out and up over the hill, thence flowing by gravity to the pumping station. On account of the difference of specific gravity of the oil and water, the oil will rise to the level of the water reservoir while the water in the oil line will not. Hence when practically all the oil has passed over the hill the flow stops automatically. It therefore requires no watching or operating except to fill the oil tank and turn on the water pressure. The tank is drained of water each time before being filled again with oil.

Cracks in the Harvard Stadium, recently reported in the daily press, are merely shrinkage cracks which occurred years ago. Some 6 x 6-in. timbers then placed as an extra precaution have remained and only recently have attracted attention.

A Deep-Water Dredge Bucket.

A special clam-shell bucket of 2 yd. capacity for excavating very hard material at a great depth below the surface of the water has recently been designed and successfully operated in digging the trench for the steel and concrete tubes of the double-track railroad tunnel under the Detroit River. The bucket is about 4 ft. wide over all, about 12 ft. long and 7 ft. high when open, about 13 ft. high and 8½ ft. long when closed. It weighs 16,300 lb. and, except for the plates and pins, is made of cast-steel and is fitted with four operating ropes or the bights of two ropes arranged in two pairs 2 ft. apart leading to the point of the boom and preventing the bucket from twisting.

The operation of the bucket is very simple and is effected, as may easily be traced in the engravings, by means of two principal power levers and connecting links together with the sheaves and lines which allow the bucket to open by gravity and develop a very powerful force to close it simultaneously with the excavation of the earth with which it is filled.



Deep-Water Excavating Bucket Showing Operating Links and Sheaves.

Some of the material excavated was a very refractory clay called gumbo, or sky-pitch, which is extremely hard to dig and is likely to adhere to the bottom of ordinary buckets. For this reason it was necessary to have great closing power and because it was anticipated that it might encounter logs or other large obstructions, it was given extra strength by providing a factor of safety of 8 or 10 for the principal working parts under normal load. The weight of the bucket and contents was too great to be operated by a single line with the engine used by the contractors, and in order to avoid the use of a burden block the bucket was designed as shown with a double purchase on the power arm, thus allowing one end of the line to be made fast to the derrick boom and providing a two-part tackle without complications.

The corner arms have an I-shape cross section and are believed to be the first ones made from cast steel for such use. The 6-in. hinge pin is pinned to the side pin lugs on each side, holding the hinge scoops perfectly rigid and keeping them continually in alignment. All other pins and shafts are 4 in. in diameter with bronze bushed bearings. The 24-in. sheaves are thoroughly guided to prevent the lines from becoming disengaged and the scoops are made with a 12 x 1-in. lip and a ¾-in. back plate and have 9 cast-steel teeth.

Much of the excavation being done on a steep slope at a great depth, it is sometimes difficult for the operator to tell when the bucket reaches the bottom and it often tips over. When this happens the cables are prevented from becoming entangled by means of a circular guard and small rollers on top (not seen in the illustrations), which have been found very useful and successful.

The bucket has been used to a depth of 78 ft. and has endured very severe service, and at one place where the excavation broke through the walls of the shore tunnel it pulled out large quantities of 12 x 12-in. timbers, railroad rails and a considerable amount of steel sheet piling. One section, about 10 ft. wide and 26 ft. long, was very badly twisted and required a very heavy pull to remove it.

The bucket was designed and manufactured by the G. H. Williams Co., Cleveland, Ohio, for Dunbar & Sullivan, sub-contractors for the excavation. The Butler Bros. & Hoff Co. are the contractors for the construction of the tunnel.

Book Notes.

In a 448-page book of pocketbook size, entitled "Concrete," Mr. Edward Godfrey has brought together a number of articles contributed by him to engineering journals, together with a large amount of information relating to various aspects of concrete. The introductory chapter is a survey of the field of concrete design and construction in which the author criticizes many theories and methods of computation that have been advocated by other engineers. The following 162 pages relate to cement, lime, sand, aggregates, mortar, concrete, steel for reinforcement, handling and placing concrete, the setting and hardening of concrete, finishing concrete surfaces, forms, the properties of concrete, general features of design and construction, estimating costs and the standard specifications for natural and Portland cement adopted by a number of engineering societies. The next section of the book, comprising 76 pages, is reprinted from "Engineering News," and contains discussions by the author of the design of reinforced concrete beams and slabs, the design of reinforced concrete columns and footings and the design of reinforced concrete retaining walls, together with all the correspondence in that journal relating to them. The third section of the book, which occupies 156 pages, is a collection of articles contributed by the author to "Concrete Engineering," together with the discussions of them in that journal. The articles relate to reinforced concrete engineering, the design of reinforced concrete beams and slabs, the design of reinforced concrete arches, the design of foundations, the shear of concrete and its bearings on the design of beams, the design of reinforced concrete columns, the design of dams and the use of concrete in them, the design of reinforced concrete chimneys and the design of domes, vaults and conical coverings. The final section is a collection of 31 full-page illustrations of various concrete objects or structures. A large part of the book deals with topics regarding which there are differences of opinion. The author states his own views regarding them and explains his reasons for holding them and frequently criticizes contrary opinions in detail. (Pittsburg: Monongahela Bank Bldg., Edward Godfrey, $2.50.)

The high-speed passenger elevator is responsible for the remarkable development of tall office buildings as well as for the creation of important minor types of business buildings, yet less has been written about the relations that the service rendered by an elevator equipment of a building bears to the convenience of access to its upper floors and its consequent success commercially than upon any other phase of vertical transportation. Hence the book of Mr. Reginald P. Bolton, on "Elevator Service," which is a pioneer effort in treating of the operating conditions and proportions of passenger elevator plants, should meet with a favorable reception and may be studied by architects, engineers and those having to do with the design and proportioning of such plants to buildings of particular size and character. The book discusses in considerable detail the various features of the problem of elevator service as it is affected by the floor area of the building in relation to its height, the characteristic operating conditions that ordinarily obtain in business buildings and the effect of the personal equation of both operators and passengers upon the service, and many suggestions are offered as a result of careful studies of the problem. As a method of rating the work of the elevator, Mr. Bolton suggests a standard rating based on an average passenger travel each way per trip of that number of persons which corresponds to four-tenths of the number of floors served, which is shown by examples to provide for an average service very close to the maximum hourly capacity, while still introducing a small margin for excess capacity to provide for unusual conditions of travel. The book concludes with a discussion of express services, the shape and size of the elevator car, load and speed combinations and the service proportionate to the building, with diagrams, formulas and tables for passenger travel, schedules, express operation, etc. A chapter is added containing definitions of some terms used in connection with elevators. (Published by the author, 527 Fifth Ave., New York, $5.)

Letters to the Editor.

THE NATIONAL FORESTS IN THE WHITE AND SOUTHERN APPALACHIAN MOUNTAINS.

SIR: Unwillingness on the part of the leaders of Congress to permit legislation at this session beyond the regular appropriation bills works particular hardship in the White Mountains. The protective forests on the high slopes have been going at the rate of 300 acres daily or 25,000 annually, which affects the agricultural, manufacturing and navigation interests throughout New England.

The bill for National Forests in the White Mountains and the Southern Appalachian Mountains has been indorsed by the entire country. Lumbermen and paper-makers, farmers and business men and the press, without exception, have urged this measure for five years. Two years ago it passed the Senate and was recommended unanimously to the House of Representatives by the Committee on Agriculture, but owing to the opposition of the Speaker did not then become a law.

At a recent hearing before the present Committee on Agriculture, the governors of Georgia and New Hampshire, with official delegates from the governors of twelve other states and a group of two hundred business men and citizens from all of the states east of the Mississippi, except four, presented evidence that the timber supply is disappearing, that water powers are affected, that navigation is crippled and that agriculture in the low lands, particularly in the south, is, in many cases, wiped out by floods. Delegates from the Pittsburg Chamber of Commerce showed that recent floods cost that city ten millions of dollars. Lawrence, Mass., sent a delegation, including the Mayor and the President of the Board of Trade, to show the importance of the Merrimac for drinking water, as well as for power. To all these Congress apparently has turned a deaf ear.

When the matter came before the Judiciary Committee the governors of Massachusetts, Connecticut and North Carolina sent able lawyers to Washington to prove that the matter is one in which Congress has without doubt power to act.

Neither the Committee on Agriculture nor that on Judiciary has made any report and the impression prevails at Washington that they do not intend to do so. Is there any reason why the leaders in the House of Representatives should longer obstruct the will of the people? When six New England states and eight Southern states unite in asking prompt action by Congress do the men who guide that body propose to turn the matter calmly down? What do the representatives from the South and from New England intend to do about it?

Very respectfully yours,
F. W. ROLLINS.

Concord, N. H., April 4.

CHROME PIGMENTS FOR PAINTS.

SIR: In a recent publication under date of May 10, 1907 (Bulletin No. 30, Office of Public Roads, U. S. Department of Agriculture), entitled "The Corrosion of Iron," the writer discusses the possibility of using certain inhibitors in the priming coats of paints and other protective coverings. The suggeston was publicly made that slightly soluble chromates should be theoretically the best protectives to apply to iron and steel surfaces. Numerous chrome pigments have been tested by the writer in reference to their inhibition value, the work having been done in large part previous to the publication of the bulletin above cited. Owing to included impurities many of the commercial chrome pigments have been found to stimulate rather than inhibit corrosion, and the use of these for such purposes should be carefully guarded against. It has been found, however, that zinc chromate and a pigment made by precipitating barium and calcium chromates in molecular proportions give excellent results in the absence of impurities, such as sulphates, chlorides, etc. Prussian blue has also proved itself among the best of the rust inhibitors, so that excellent formulæ can be devised for good greens, using the above pigments with small amounts of pure calcic carbonate and magnesium silicates, etc. Some of the basic orange chromates have also given good results.

It has been reported that patents have recently been applied for on a combination of zinc chromate with linseed oil as an inhibitive coating for iron and steel. The details of the claim cannot yet be known, but in view of the general publication of the writer's results, it does not seem that the grant of such a patent would be justified. It is the policy of this department to give out the information it obtains for the free use of every one in the country, and particularly to safeguard the interests of the farmers. It would be a misfortune, in case these inhibitive formulæ prove themselves of high protective value, that their general use should be tied up by individual patent claims.

Respectfully,
ALLERTON S. CUSHMAN.

Office of Public Roads, U. S.
Dept. of Agriculture, April 8.

THE HEATING OF PASSENGER TRAINS hauled into New York City on the New York Central Railroad by electric locomotives is done by small oil-fired vertical tubular steam boilers in the cabs of the electric locomotives. The boilers are short vertical shells, each containing 800 ½-in. copper tubes, which are mounted on cast iron brick-lined fire-boxes. Kirkwood burners and ordinary commercial fuel oil, carried in a 40-gal. tank in the cab, are used. The feed water, carried in a 200-gal. closed tank, is kept under 135-lb. air pressure from the air brake compressor, which enables it to be fed to the boiler against its working pressure of 80 lb. without pump or injector. The atomizing burner is started with air pressure but later thrown over to steam when the pressure is raised.